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ERRATUM

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Page 471, footnote, for "1917", read "1907".

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Paper No. 1426

THE ECONOMICS OF STEEL ARCH BRIDGES*

By J. A. L. Waddell, M. Am. Soc. C. E.

With Discussion by Messrs. Paul A. Blackwell, T. Kennard Thomson, Charles Evan Fowler, W. B. Farr, Horatio P. Van Cleve, Henry S. Jacoby, H. S. Sales, David A. Molitor, Edward W. Stoney, F. H. Frankland, and J. A. L. Waddell.

Synopsis.

Up to the present time, nothing at all certain has been known concerning the economics of steel arch bridges, the weights of metal required to build them, or how they compare in cost with corresponding steel truss bridges.

The objects of this paper and its anticipated discussions are to settle finally every important economic question that can arise in the designing of steel arches; to give formulas and diagrams for determining, with a fair amount of accuracy, the weights of metal in both arch bridges as a whole and the arches themselves; and to indicate the relations between the weights and costs of arch bridges in comparison with those of corresponding truss bridges.

There are eight economic problems set for solution, and all of them have been solved—the first two by using certain formulas given in the writer's new book, "Bridge Engineering", and the other six by a large number of special arch designs and the resulting estimates of weights of metal. Incidentally, during the investigation, there have arisen and have been solved a few minor questions which may properly

* Presented at the meeting of May 1st, 1918.

be termed side issues. The computations have been made for both railway and highway arch bridges; and the weights of metal are plotted for both carbon-steel and nickel-steel structures. Several diagrams are given to show the percentage effects of weight increase due to departure from economic conditions.

The need for the special knowledge concerning arch bridges presented in this paper has been recognized during the last three decades by American structural steel engineers; but the authors of books on bridges (the writer included) have hitherto avoided the issue because of the immense amount of work involved in the solution of the various problems. The writer has tried of late years on several occasions to persuade some of his brother engineers to undertake the task; but, not meeting with any success, he finally decided to do the work himself. The results of his efforts are herewith presented to the members of the Society with a most earnest request for a thorough discussion, particularly by those who specialize in bridgework.*

Concerning the economics of steel arches, very little, indeed, is known, the reasons being that comparatively few such structures have been built in America, and that the subject of economics in bridge design is a study to which, generally speaking, but slight attention has been paid until late years. Occasionally, an American bridge engineer builds a steel-arch structure; but usually simple-truss spans are adopted because their approximate weights and their economics are on record. The writer is of the opinion that if more were known generally concerning the weights of metal in steel-arch structures, their economic functions, and their comparative economy in respect to the corresponding simple-truss structures, the esthetic type would be adopted more frequently, in spite of the greater labor it involves in both designing and construction. An arch is almost always a handsomer structure than an ordinary truss bridge; and the time has

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* As it would be out of the question to publish the great mass of graphic calculations of stresses, diagrams of stresses and sections, tabulated estimates of weights of metal, and other computations which were prepared specially for this paper, copies of them are filed in the Engineering Societies Library, and as the original dimensions of the paper have been reduced in the interest of economy of publication, the memoir as first written will also be so filed.

The writer desired to quote three pages from the chapter relating to Arch Bridges from his book "Bridge Engineering", giving ten equations which form the basis of the solution of the first two problems discussed, and an explanation of how they were established, but this was prevented by the requirements of the Publication Committee. It will be necessary, therefore, for the reader to refer to the author's book in order to comprehend the *modus operandi* of solving the said two problems.

arrived for all American engineers to pay proper attention to the important feature of esthetics in the preparation of designs.

The economic problems concerning the design of steel arch bridges which still require solution are:

First.—The economic ratio of rise to span length;

Second.—The economic depths for the ribs;

Third.—The economic location for the crown-hinge in three-hinged, spandrel-braced arches;

Fourth.—The ratios of weights of metal required for the solid-rib, the braced-rib, and the spandrel-braced types;

Fifth.—The ratios of weights of metal required for the hingeless, the two-hinged, and the three-hinged types;

Sixth.—The economics involved by making arches three-hinged for the dead load and two-hinged for the live load;

Seventh.—The economy of the cantilever arch with suspended end spans, as compared with an ordinary arch and two flanking simple-truss spans; and

Eighth.—The ratios of weights of metal required for certain portions of arch bridges as compared with the corresponding portions of simple-truss bridges of the same span and the same live-load carrying capacity.

Problem No. 1.

Up to the present time, no reliable information has been available concerning the economic ratios of rise to span length, a few computers recognizing that it lies between the limits of 15 and 30%; but no systematic investigation of the question, as far as the writer knows, has ever been made.

However, by the use of ten equations* for three-hinged arches, the weights of metal for both the solid-rib and the braced-rib types of various span lengths, and having any ratio of rise to span, any depth of rib, and any live load or dead load, can be found readily and with considerable accuracy. The formulas given by these equations are based on the writer's specifications for designing†; which specifications, it is generally acknowledged, are substantially in accordance with the best American bridge engineering practice. From these equations, by

* Given in Chapter XXVI of the writer's "Bridge Engineering."

† "Bridge Engineering", Chapter LXXVIII.

the substitution of certain assumed values, can be found both the economic ratios of rise to span length and the economic depths of arch ribs.

In order to cover effectively the ordinary range of span lengths for arch designing, it was necessary to compute for three lengths, *viz.*, 200, 500, and 800 ft.; and, in order to include all ordinary structures, it was decided to determine the economic functions for both steam-railway bridges and combined highway and electric-railway bridges. For steam-railway bridges, double-track structures to carry Class-60 live load were adopted; and for combined highway and electric-railway bridges there was chosen a deck, 60 ft. wide from out to out, having at the middle a double-track electric railway, on each side of the latter an 11-ft. clear space for vehicles, and on the outside of each roadway an 8-ft. sidewalk, the entire roadway, including the track space, being paved with creosoted blocks resting on a reinforced concrete base, and the sidewalks being of reinforced concrete slabs.

The work involved in the investigation was, of necessity, reduced to a minimum, being great enough, in all conscience, as it was; hence solid-rib arches were computed for spans of only 200 ft., because structures of that type of longer span would be uneconomical. In some cases, owing to the cut-and-try method which had to be adopted, many assumptions of ratios of rise to span were tested; but, in order to save space, the unnecessary results have not been recorded, three only for each case serving the purpose of determining the minimum.*

In finding the weights of metal, it was necessary first to assume the depths of the arch ribs, and afterward to test them for economy; but, fortunately, in every case the first guess was almost exactly correct. These checks on the economic depths of arch ribs are discussed hereinafter.

In computing the economics of arches, it is not sufficient to ascertain merely the weights of metal in the arch ribs, for it is necessary to take into consideration also the weights of metal in the superimposed columns and the vertical sway bracing between them; and in comparing the weights of arches with those of the corresponding simple trusses, the former must be increased by the weights of such columns and their vertical sway bracing. In general, it may be assumed that the weight of the entire lateral system in a simple-truss bridge will

* See manuscript on file in the Engineering Societies Library.

offset that in the bracing of the floor system, plus that of the rib flanges of the corresponding arch structure, plus that of the other diagonals between the arch ribs. The error involved in this assumption is so small that it will not affect the correctness of the comparisons made in this investigation.

The problem of determining the economic ratio of rise to span is not as simple as it might appear at first thought; for it is a compound one, being dependent on the conditions precedent. These may be divided into four cases, *viz:*

First.—When the rocky sides of the chasm to be spanned are practically vertical, with the grade line about tangent to the crown of the top chord of the arch. In this case, as previously indicated, the weight of the superimposed columns and their bracing must be considered with that of the arch rib in determining the economic rise.

Second.—In the case of a half-through structure, where the variations in the sum of the weights of hangers and columns are comparatively slight for changes in rise, and where, consequently, the economics are dependent mainly on the weight of the arch rib alone.

Third.—When the grade line is far above the crown of the arch, and, consequently, the greater the rise the less the weight of the superimposed columns and their bracing. This is just the opposite condition to that of the first case, because, in that one, the greater the rise the greater is the average length of the spandrel columns.

Fourth.—When the chasm is decidedly V-shaped. In this case the grade line may be either tangent to, or well above, the crown of the top chord.

These four cases have been considered separately.*

Case No. 1.—From Table 1 it may be concluded that for braced-rib arches the economic ratio of rise to span for all spans between 200 and 800 ft. may be taken at 0.225, without making any appreciable error. The calculations, however, show that within the limits of 0.2 and 0.25 there is very little variation of arch weight. In many cases one cannot exercise much choice in the selection of the rise for an arch, because the profile of the crossing and the adopted grade

* In the original memoir are given the designing data and the tabulated results of substitution in formulas for seven cases of both railway and highway bridges and covering both solid-rib, and braced-rib arches, also a few notes concerning certain short cuts that were used in the computation. Each table contains three records, the middle one indicating the economic ratio of rise to span, because it records the minimum weight of metal, in pounds per linear foot per side.

TABLE 1.—Summary of Results of Economic Rise Investigations for Braced-Rib Arches.

Span, in feet.	Economic Ratios of Rise to Span Length.	
	Steam-railway structures.	Combined highway and electric railway structures.
200	0.225	0.225
500	0.225	0.225
800	0.240	0.240
Averages	0.230	0.230

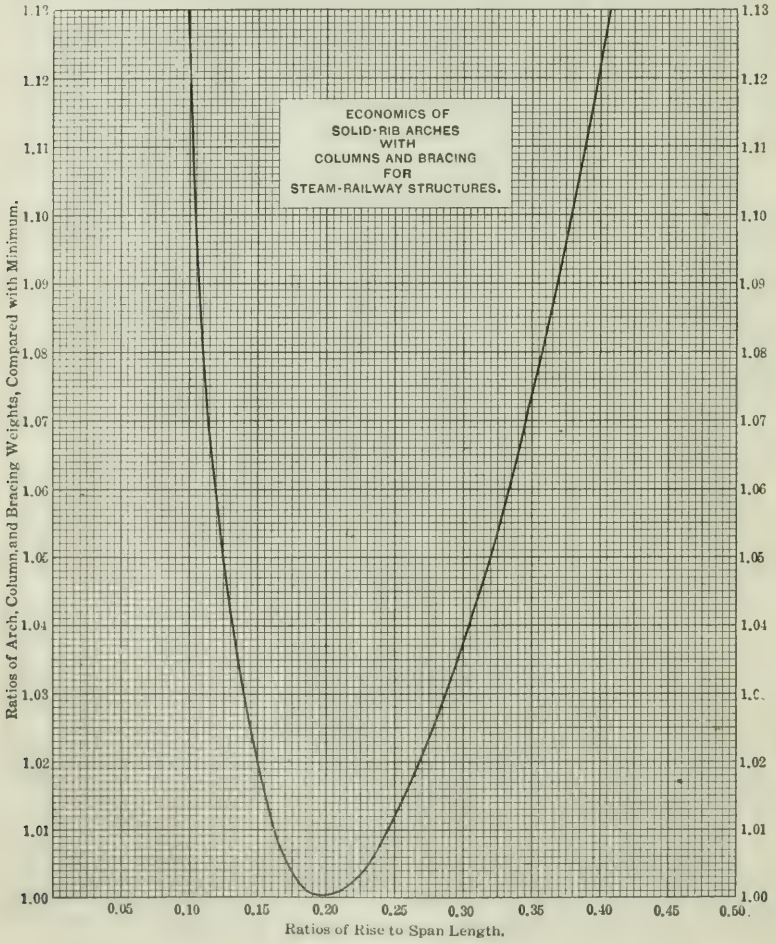


FIG. 1.

line of floor will be the governing factors. Under such conditions one should use a ratio as near to the economic one as practicable. Fortunately, as far as the matter of esthetics is concerned, such ratio is not a vital feature; and the calculations show, as just indicated,

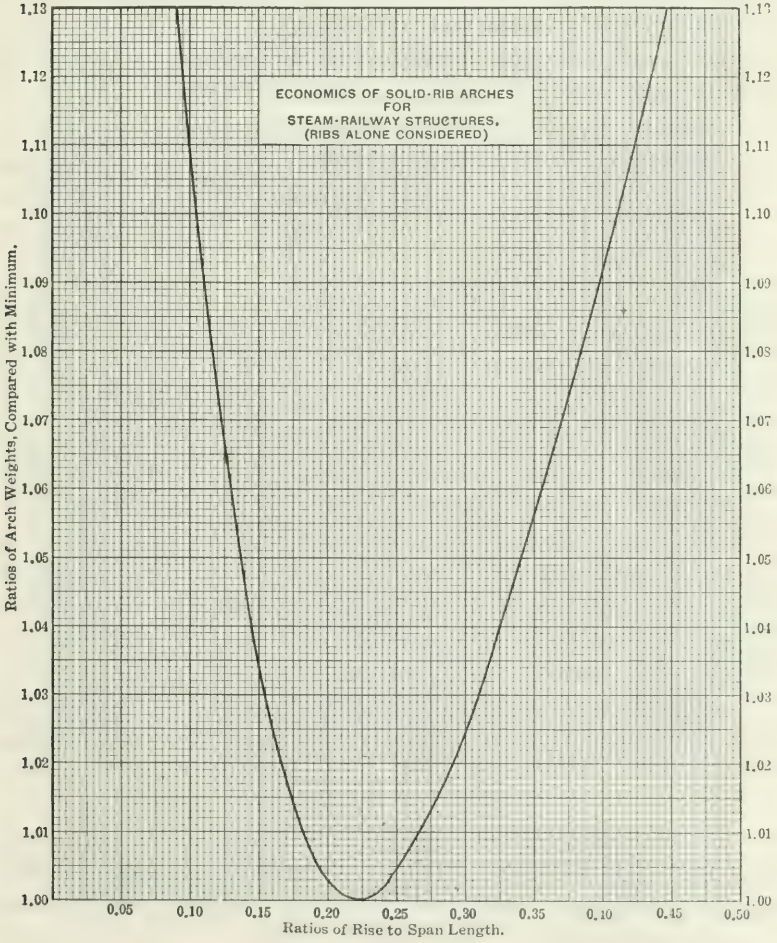


FIG. 2.

that it can be modified considerably without greatly augmenting the total cost of the structure.

For solid-rib arches, the most economic ratio of rise to span may be taken as 0.2, and the ordinary range as from 0.175 to 0.225. In order to show graphically the economics of solid-rib arch bridges,

Figs. 1 and 2 have been prepared, the former being applicable when the superimposed columns and their bracing are considered, and the latter when only the ribs themselves are under discussion. The abscissas represent the ratios of rise to span length, and the ordinates the corresponding percentages of metal required, the percentage for the minimum weight being one hundred.

Figs. 3 and 4 present the same data for braced-rib arches as are shown in Figs. 1 and 2 for solid-rib arches.

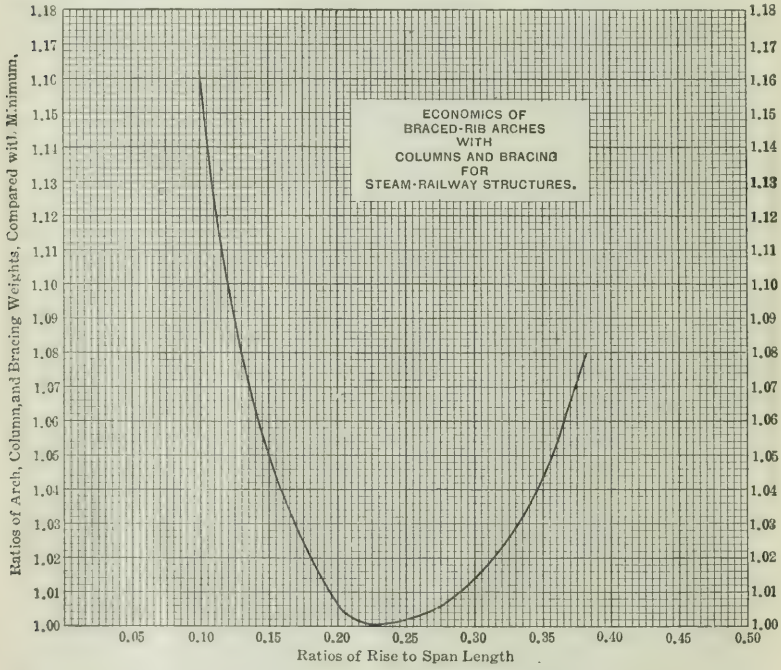


FIG. 3.

Another deduction that may be made from Table 1 is that the economic ratio of rise to span is independent of the kind of traffic which the structure carries; and the calculations have indicated that it is independent also of the size of the loading.

Case No. 2.—A systematic extension of the economic calculations for the 200-ft. and the 500-ft. spans showed that, for the solid-web type, the economic ratio of rise to span (when merely the weight of the arch itself is considered) is 0.225; and for the braced-rib type it is 0.3. A variation of 0.025 in either direction, however, does not greatly increase the weight of metal.

Case No. 3.—When the grade line is well above the crown of the arch, as shown in Fig 5, it is evident that the greater the rise, the less will be the average length of the superimposed columns and

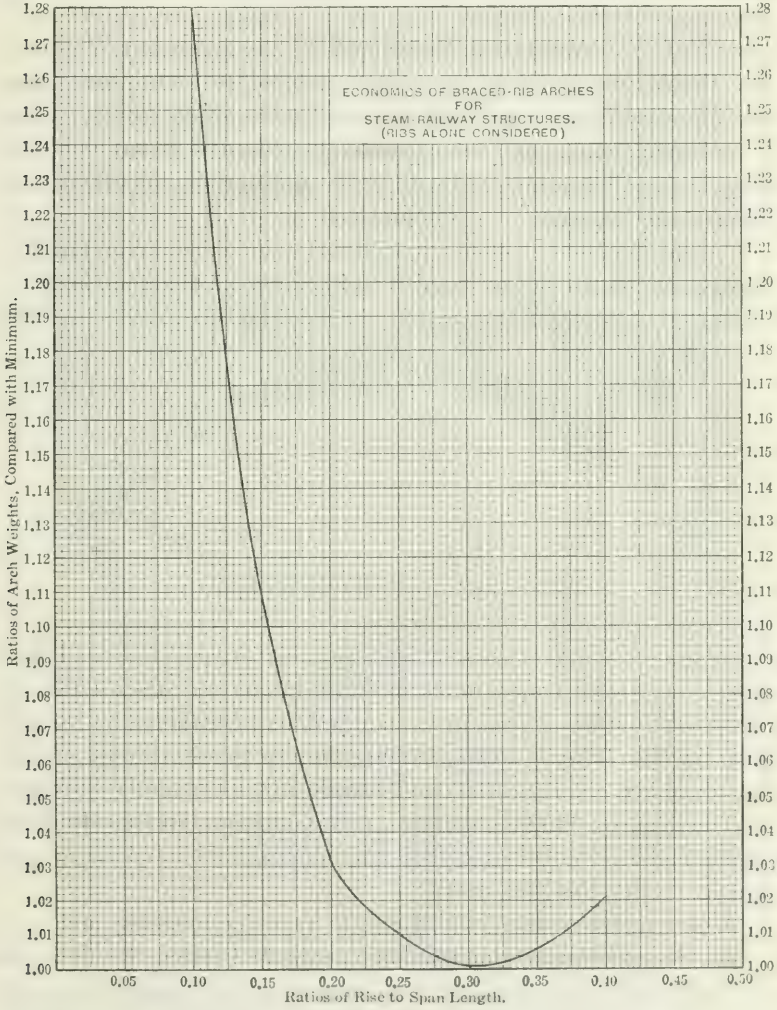


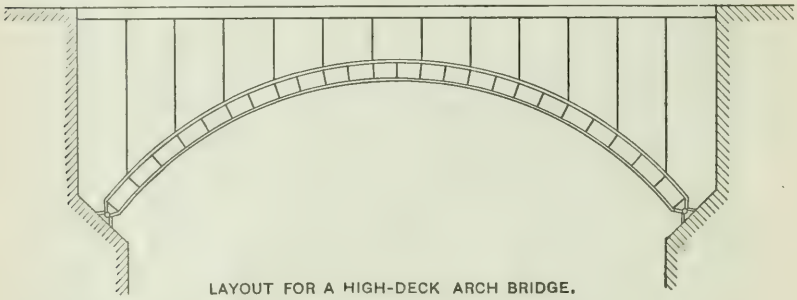
FIG. 4.

bracing—also that the greater the vertical distance between crown and grade the greater the saving in metal weight per foot of vertical reduction in height of columns. This last consideration, though, is of minor importance, and would affect very slightly, indeed, the

economic ratio of rise to span. For the solid-rib type, that ratio for high-decked arches (those where the grade is well above the crown) will vary from about 0.25 to 0.28, and for the braced-rib type from about 0.35 to 0.38.

Case No. 4.—Where the rocky sides of the cañon are V-shaped, there is only one way to determine the economic location of the springing points, and that is by trial; but the results of the investigations in this paper will enable the computer to reduce to a minimum the number of trials required, and will cut down materially the calculations necessary for the solution of the problem.

For spandrel-braced arches it will suffice for economy of metal to make the rise of the bottom chord from 20 to 25% of the span length.



LAYOUT FOR A HIGH-DECK ARCH BRIDGE.

FIG. 5.

In respect to the economic ratios of rise to span for two-hinged arches, there is no reason which militates in any way against the correctness of the assumption that they will be practically the same as those for three-hinged arches, because the two types are very similar; and, moreover, they are likely to be made still more similar in future constructions by hinging the crown at mid-depth for the dead-load stresses and afterward making both the upper and the lower chords continuous for the live-load stresses.

As for the economic ratio of rise to span for hingeless arches, in making the calculations for the 500-ft. span in Problem No. 5, it was assumed to be the same as for three-hinged arches, computing in both cases along the mid-line of the rib; but some special supplementary calculations, involving an entirely new layout and design, showed that it is somewhat greater. It may be taken as 0.28 when the grade line is about tangent to the crown of the upper chord, as 0.33 for half-through arches, and as 0.38 for high-deck structures.

PROBLEM No. 2.

As far as solid-rib arches are concerned, the question of the economic depth of the rib does not arise; because that dimension should always be made as great as a proper consideration of the section for resisting compression will permit. The member acts as both a beam and a strut; hence, if the webs are made thin and of economic depth for the bending, they will not be thick enough to comply with the rules governing the proportioning of built compression members. It is true that rigid adherence to these rules is not always necessary in arch designing, but they should never be transgressed to any great extent.

For braced-rib arches, however, it is practicable to use the economic rib depth. Up to the present time, no rule for determining this has been given. Fortunately, No. 8 of the before mentioned ten equations offers a means for solving the problem. The area of the chords,

$$A_c = \frac{T + \frac{2M}{d}}{p_c},$$

is divided into two parts, *viz.*, $A_t = \frac{T}{p_c}$, due to direct thrust, and $A_m = \frac{\frac{2M}{d}}{p_c}$, due to bending from the live load only, provided that the center line of the rib is properly located; in which case the area for the web, $A_w = \frac{2S}{p_w}$, is also due to live load only. On that basis the partial weight of the chords, A_m, should equal the weight of the web, in order to obtain the greatest possible economy of metal. After determining the values of A_m and A_w, if they are found to be unequal, the economic depth, d', can be obtained thus:

$$A'_m = A_m \times \frac{d}{d'},$$

$$\text{and } A'_w = A_w \times \frac{d'}{d}.$$

Equating these makes $A_m \frac{d}{d'} = A_w \frac{d'}{d}$, or $d' = d\sqrt{\frac{A_m}{A_w}}$

Replacing A_m and A_w by their values gives:

$$d' = d\sqrt{\frac{\frac{2M}{d\,p_c}}{\frac{2S}{p_w}}} = d\sqrt{\frac{M\,p_w}{S\,d\,p_c}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

After making the calculations to determine the economic ratio of rise to span for each case, the assumed economic rib depth was checked by Equation (1), the results being as shown in Table 2.

TABLE 2.

Span, in feet.	Assumed depth, in feet.	Economic depth, in feet.	Ratio of economic depth to span.
200	16	15.48	0.0774
500	40	38.65	0.0773
800	60	60.24	0.0753

It is evident from Table 2 that the economic depths of braced-rib arches vary from about 7.5 to 7.75% of the span; but no mistake of any importance would be made if they were taken as 7.5% for three-hinged arches of all span lengths. There is no reason for deeming that the economic depth is any different in the case of two-hinged arches; and in hingeless arches it is probable that the economic average depth will be about the same. In spandrel-braced arches, it will suffice to make the arch depth at mid-span equal to from 6 to 8% of the span length; but here esthetics—not economics—should govern.

In two-hinged arches of very great span length, such as the famous Hell Gate Arch Bridge, it is proper to consider the economics involved by varying the arch depth approximately in proportion to the size of the bending moment. Such treatment would place the greatest depth at the haunches or quarter points; and in certain cases of long spans this might be permissible, from the esthetic standpoint, but generally it is not. Crescent-shaped, two-hinged arches are economic in the feature of reducing the rib depth gradually in the outer quarters of the span; but to the untrained eye they suggest an element of weakness. Of course, the real strength is dependent solely on whether sufficient sectional areas of members, with the metal properly disposed, have been provided; but the element of stiffness is certainly not improved by making the ribs shallow—and rigidity is often just as essential as mere strength. Making the rib depth proportional to the bending moment involves uniformity in the chord sections; but this is no important desideratum. In designing arch bridges, esthetics and rigidity should never be sacrificed for the sake of economy in weight of metal, nor for convenience in manufacture.

The before-mentioned tests prove that the economic rib depth is the same for both railway and highway structures, and that it is almost entirely independent of the size of the live load.

In order to indicate the effect on the total weight of metal by varying from the economic rib depth, take the case of the 800-ft. span, in which the assumed depth of 60 ft. was almost exactly correct, and test the increase for other depths. In the railroad structure, taking the values of A_m and A_w equal to 177 sq. in. and assuming various depths, we have the results given in Table 3, from which

TABLE 3.

Assumed depth, in feet.	Ratio of assumed depth to economic depth.	$A_m + A_w$, in square inches.	Corresponding increased percentage of total weight of ribs.
30	0.50	442.5	13.0
35	0.58	406.7	7.7
40	0.67	383.5	4.3
45	0.75	368.7	2.2
50	0.83	359.9	0.9
55	0.92	355.3	0.2
60	1.00	354.0	0.0
65	1.08	355.1	0.2
70	1.17	358.2	0.6
75	1.25	362.9	1.3
80	1.33	368.7	2.2
85	1.42	375.7	3.2
90	1.50	383.5	4.3

it is evident that, for the particular span under consideration, a material variation from the economic depth of rib may be made without greatly increasing the weight of metal in the arch. Any variation that is likely to be adopted will be a reduction; because, generally, the appearance of the arch is improved by making its rib depth smaller than the economic one. The reason the percentages in the table do not increase faster with the variation in depth from the most economic is that, for this span, the area of chord section affected by the bending moment is only about one-third of the total area. The writer recognizes that this demonstration is open to criticism, because he has neglected the increase in chord sections due to the augmented dead load; but the amount thereof is immaterial.

The results in Table 3 are shown by diagram in Fig. 6. Although it is true that this diagram was prepared for spans of 800 ft., it can be used, with sufficient accuracy for all practical purposes, in the case of arches of other span lengths, its main purpose being to show about

how far in any case it is advisable to depart from the economic rib depth, which, as previously indicated, is 7.5% of the span length.

It might be well to point out that the shorter the span the greater will be the percentage of increase in weight of arch rib, for the same ratio of actual to economic rib depth. For instance, in Table 3, which

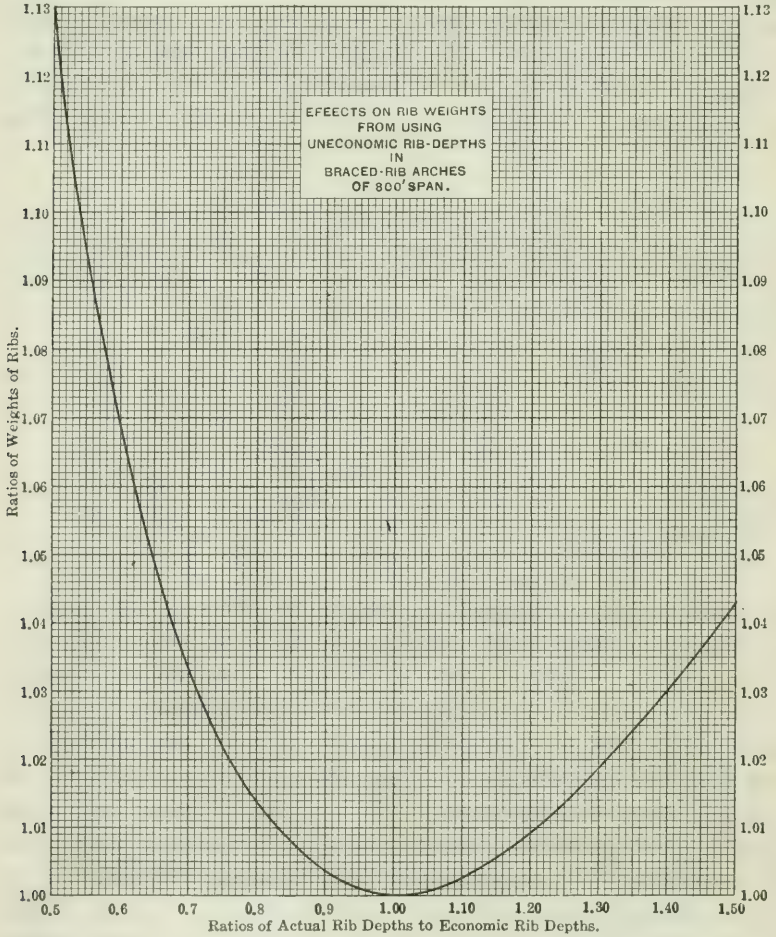


FIG. 6.

is for an 800-ft. span, if the depth is made 75% of the economic one, the increase in weight of rib metal will be 2.2%; but, in a 500-ft. span, it would be about 2.4%, and about 2.8% in a 200-ft. span. These figures endorse the statement previously made that it is proper to use the diagram, Fig. 6, for all span lengths.

PROBLEM NO. 3.

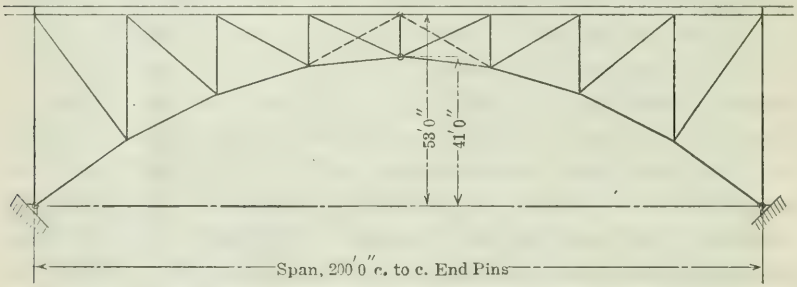
Strictly speaking, a spandrel-braced arch bridge is a structure in which the dead-load thrusts and the full live-load thrusts are carried directly by the bottom chords of the arches, the top chords and the web members serving only to brace the bottom chords under unsymmetrical or partial loading. The spandrel members thus have only a stiffening function to perform, similar to that of the stiffening truss of a suspension bridge—hence the name, "Spandrel-Braced Arch". In accordance with that definition, the crown-hinge in a spandrel-braced arch should, from the theoretical point of view, be in the bottom chord, thus obviating the necessity of the stresses from dead load and full live load traveling through the spandrel members. The simplest and most direct stress path, *viz.*, through the bottom chord, apparently should afford the maximum economy; but, in light bridges, some of the top-chord members may have sections unavoidably large in respect to the theoretical requirements; and in that case the placing of the hinge in the top chord would tend to utilize a certain quantity of idle metal, and, at the same time, reduce the sectional area of a portion of the bottom chord, thus effecting a saving in the total quantity of metal in the structure. In the writer's opinion, this is a legitimate economic expedient; but he is opposed to putting the crown-hinge anywhere except at mid-depth of rib in the case of both solid-rib and braced-rib arches. The object of this last suggested arrangement is to divide the thrust equally between the upper and lower chords of the rib; then, by keeping the center line of the latter as nearly as may be on the arc of a parabola, the chord stresses due to direct thrust will tend to divide themselves equally between the two chords, provided, of course, that the dead load is uniformly distributed. The curve of the bottom chord of a spandrel-braced arch, passing through the three hinges, should be a parabola, if the dead load is practically uniform across the span; otherwise, this curve should correspond to the equilibrium polygon for

$$\text{D. L.} + \tfrac{1}{2}\ (\text{L. L.})$$

covering the whole span.

In order to settle this question of the best location of the crown-hinge in the spandrel-braced arch, and at the same time to compare the weights of spandrel-braced arches with those of the corresponding solid-rib and braced-rib arches, two designs have been made for a

200-ft. span, spandrel-braced, steam railway, arch bridge, having the same vertical distance between top-chord and springing points as in the other two cases, as well as the same panel length and the same live load. For the purpose of avoiding the adoption of an uneconomic ratio of rise to span length, the arch depth was reduced to 12 ft. at the crown. With a depth of 53 ft. at the springing points, this provides a neat layout, as shown in Fig. 7; and the ratio of rise to span, measured on the bottom chord, is 0.2, which is about the inferior limit of the economic range. With the crown-hinge in the top chord, the ratio of rise to span length is 0.265, which is a trifle above the superior limit of the said range. These two layouts give a fair comparison between the structure with the bottom-chord crown-hinge and that with the top-chord crown-hinge. In this case there is no need of



LAYOUT FOR A 200-FOOT, SPANDREL-BRACED, STEAM RAILWAY, ARCH BRIDGE.
(Dotted lines represent reversed position of the diagonals at the crown when the hinge is located in the top chord.)

FIG. 7.

including the weights of the lateral systems in the comparison, as they are the same for each structure. The results of the calculations show that the weights of metal per linear foot of span per side (for arches only) are 1 182 lb. with the hinge in the bottom chord and 1 146 lb. with the hinge in the top chord. Contrary to what might be anticipated from purely theoretical considerations, placing the hinge in the top chord produces a less total weight of arch metal than placing it in the bottom chord, the difference in this case being about 3 per cent. A study of the two diagrams of stresses and sections shows that this result is due primarily to the effect of reversing stresses. Had the latter been ignored, the economics would have been reversed, and the difference would then have been 5% in favor of the location in the bottom chord. The provision for reversal of stresses is respon-

sible for 10% of the total weight of metal with the hinge in the bottom chord, and for only 2% thereof with the hinge in the top chord.

In the solution of Problem No. 7, it became necessary to compute the weight of metal for a 500-ft., spandrel-braced, three-hinged arch; and advantage was taken of this to find the effect on the total weight of arch metal by placing the crown-hinge first in the top chord and then in the bottom chord. The ratio of weights of metal proved to be 1.04, the economics being in favor of the higher hinge.

These results agree in kind, but not in amount, with the subsequently mentioned findings of Merriman and Jacoby in their investigation of the economics of the Niagara Arch Bridge. It may be finally concluded, therefore, that in spandrel-braced, three-hinged arches the economic location of the crown-hinge is in the upper chord.

Problem No. 4.

In determining the ratios of weights of metal required for the solid-rib, the braced-rib, and the spandrel-braced types, it was thought at first that it would suffice to test for the 200-ft. span only, and to make the assumption that what holds true for that length will hold also with sufficient exactness for all other span lengths; but, later, it was necessary to compare some longer spans. The weights of the arches with the superimposed columns and their bracing have already been determined for the solid-rib and the braced-rib types; and two spandrel-braced, steam-railway, arch bridges, of the same live-load carrying capacity and having the same vertical distance between grade and springing points, have been designed, and their weights of metal have been computed. From these weights and by using certain data in the writer's latest published work on bridges, there have been computed the weights of metal per linear foot of span per side in the 200-ft. arches and the corresponding lateral system, *viz.*, 1 146 and 204 lb., respectively, making a total of 1 350 lb.

From this and other weights previously found we have the data in Table 4.

TABLE 4.

Type of arch structure (200-ft. span.)	Total weight of metal, in pounds per linear foot per side, for ribs and lateral system.
Solid-rib	1 950
Braced-rib	1 490
Spandrel braced	1 350

Had the weight of metal in the floor system and that on piers been included in each case, we should have had the weights given in Table 5.

TABLE 5.

Type of arch structure (200-ft. span.)	Total weight of metal, in pounds per linear foot of structure, including metal on piers.
Solid-rib	5 500
Braced-rib	4 580
Spandrel-braced	4 330

From Table 5 it is evident that, even with a somewhat lower pound price of metal for plate-girder structures, the braced-rib type for railroad bridges of 200-ft. span is more economical than the solid-rib type; and the divergence of cost between them will augment as the span increases. It is probable that it will diminish somewhat for spans down to 100 ft. (at which length simple, plate-girder spans would be preferable); but it is not likely that, for railroad structures, the solid-rib type will in any case be quite as economical as the braced-rib type.

In making the calculations for Problem No. 7, as previously stated, it became necessary to compute the weight of metal for a 500-ft., spandrel-braced, three-hinged arch; and by a little extra computation it was practicable to compare the total weights of metal in braced-rib and spandrel-braced arch bridges for double-track, steam-railway structures of that span; but it was necessary to include the floor metal, because of the longer panels in the spandrel-braced structure. The latter required about 7% more metal than the braced-rib structure; but a revised set of computations, with more nearly equal panel-lengths, would probably have reduced that difference somewhat, although it certainly could not have lowered it more than 2 or 3% of the total weight per foot.

Concerning the economics of the spandrel-braced arch, as compared with the braced-rib structure, it appears, therefore, that it is a little lighter for the 200-ft. span and a little heavier for the 500-ft. span; but, if the arch is to be erected by cantilevering, the saving in weight of erection metal will nearly always make the spandrel-braced structure the more economical.

PROBLEM No. 5.

Up to the present time, the comparative economics of hingeless, two-hinged, and three-hinged arches have been absolutely undetermined, as is shown by the diametrically opposed views of a number of engineering writers. Merriman and Jacoby claim that, in their analysis of the two-hinged, spandrel-braced, arch bridge at Niagara, they found a saving of main sections amounting to 0.8% by the use of a crown-hinge in the lower chord, one of 8.8% by placing it midway between the chords, and one of 11.8% by putting it in the upper chord. Again, the late F. C. Kunz, M. Am. Soc. C. E., a high authority on bridges, in a letter to the writer, gave two formulas for weights of metal in two-hinged arches and their lateral bracing, and stated that for three-hinged arches such weights may be reduced 15 per cent.

On the other hand, Malverd A. Howe, M. Am. Soc. C. E., found, for a 416-ft. arch bridge, having a 67-ft. rise, the following relative weights of metal:

Hingeless type	1.00
Two-hinged type	1.21
Three-hinged type	1.30

Joseph W. Balet claims* that there is no difference in the economics of the two-hinged and three-hinged arches, but that the hingeless type is more economical than either.

Again, Willis Whited, M. Am. Soc. C. E., is quoted† as stating that "an arch without hinges, or with only two hinges at the ends, has less material and looks better than a three-hinged arch".

From the preceding it would appear that there is a strikingly great uncertainty about the economic effect of the hinging of arches; and, in consequence thereof, the writer has prepared diagrams of stresses and sections for 500-ft. arch bridges, first, for the three-hinged type to carry the steam-railway load hereinbefore specified, then a similar structure to carry the highway and electric-railway loadings, also previously indicated; second, for the corresponding two-hinged type of structure with both loadings; and, third, for the corresponding hingeless type of structure, also with both loadings.

In order to save time, the intensity for chord members was assumed for all cases to be 14 000 and that for web members 12 000 lb. The

* "Analysis of Elastic Arches."

† In Tyrrell's "History of Bridge Engineering."

former intensity is for compression only (for which kind of stress the chords were always proportioned); but the latter intensity applies to both tension and compression, because it allows about the right amount for area lost by rivet holes, and conforms for web members in compression quite accurately with the formula,

$$C = 16\,000 - 60\,\frac{l}{r}.$$

After all the diagrams of sections were completed, the correctness of these assumed intensities was checked, and the results were found to be exceedingly close, the average error for all six designs being only 1%, and the maxima 4% on the side of safety and 2% on the side of danger. Moreover, these variations affected the various types about alike, hence the approximate assumptions do not tend toward any unfairness in the comparison of weights.

There was another short cut made that involves a slight error in the total weight of web metal; but it affects all cases to the same extent. It is the use of a single value of impact for all the truss members of the span, instead of varying it for each web member according to the length of span covered by the moving load when the said member receives its greatest stress.

The weights of the main members in all six cases were computed, and the details were properly allowed for by percentage, due cognizance being taken of the weights of the special details at the hinges and of those at the anchorages. In the hingeless and the two-hinged types the temperature stresses were computed for a variation of 60° Fahr. above and 60° below normal; but when temperature stresses were added to those due to live load, impact, and dead load, the intensities of working stresses were increased by 20 per cent. As the spandrel columns and their bracing would average in length about the same for all three types, the weights of these parts of the structure were not considered, and only those of the arch-ribs were compared.

It was found that the economics of the three types depend mainly on two important features of the designing specifications, *viz.*, the method of treating reversing stresses, and the importance accorded to temperature stresses. If the effect of stress reversion is entirely ignored, the hingeless and the two-hinged structures will be lighter than the three-hinged one; but if each section on which there is reversion is computed for the greater stress plus 50% of the smaller

one, which is in conformity with many of the best modern bridge specifications, there is only a small advantage in favor of the two-hinged type over the three-hinged. Probably, if the percentage were 75, instead of 50, as some engineers specify, the weights would be alike for the two types.

If temperature stresses are entirely ignored, the two-hinged type will have an advantage of about 7% over the three-hinged type; but if they are provided for without augmenting the intensities of the working stresses, there will be a 7% advantage for the three-hinged type. If the intensities for the combination of the stresses due to all the loadings and the temperature effects are increased 20% above normal, there is an advantage for the two-hinged type of 5% or more. Probably, if the increase of intensity above normal were 10% instead of 20, the weights of metal for the two types would be about equal. As it is logical to allow a 20% increase of intensities for the combination, it is evident that, for 500-ft. spans, the two-hinged type, as far as weight of metal is concerned, has the advantage over the three-hinged. The exact figures for the comparison are as given in Table 6.

TABLE 6.

Character of structure.	Number of hinges.	Weight of metal, in pounds per linear foot per side.	Comparative percentage.
Railway	3	2 180	100
Railway	2	2 075	95
Railway	0	2 295	105
Highway	3	1 230	100
Highway	2	1 134	92
Highway	0	1 250	102

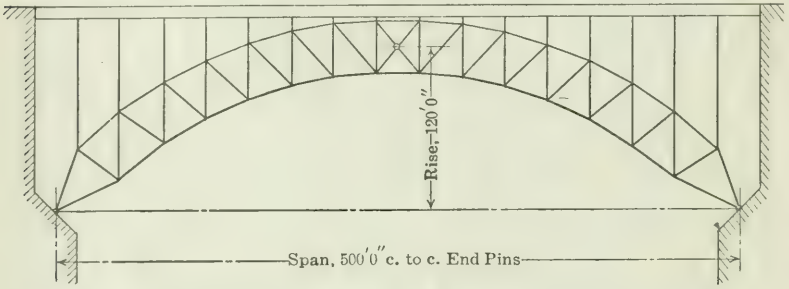
In the comparison in Table 6 the effects of wind loads have been ignored, for two reasons: first, because it is probable that they would be so small as to have no influence on the sections after allowing the usual 30% excess for the intensities of working stresses; and, second, because any small variations they might involve would be practically of like kind and amount for all cases.

Fig. 8 is the layout for the three-hinged and two-hinged arches of 500-ft. span, and Fig. 9 is that for the corresponding hingeless structure.

It might be well to call attention to the fact that in retaining the same outline for the ribs of the 500-ft., two-hinged arch as adopted

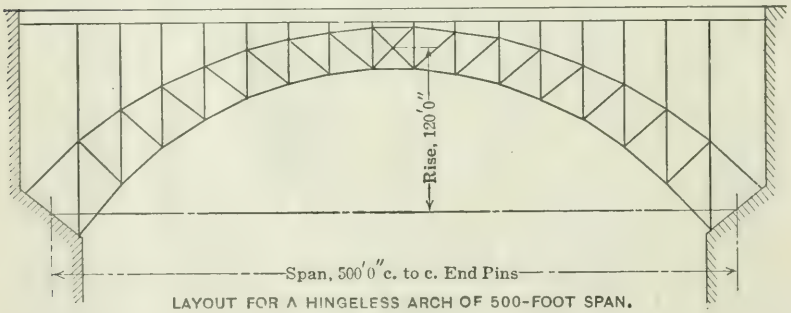
for the corresponding three-hinged structure, a slight injustice has been done to the former type, because its approximately economic shape is that of a crescent; but the difference in weight involved by this variation from the ideal is small. The reason for ignoring it is the saving in labor involved in making computations by assuming the two outlines to be identical.

Again, it is possible that the layout for the hingeless type is not the most economical one that can be drawn; but it is thought to be



LAYOUT FOR THREE-HINGED AND TWO-HINGED ARCHES OF 500-FOOT SPAN.
(In the three-hinged arch the chords in the middle panel are false members)

FIG. 8.

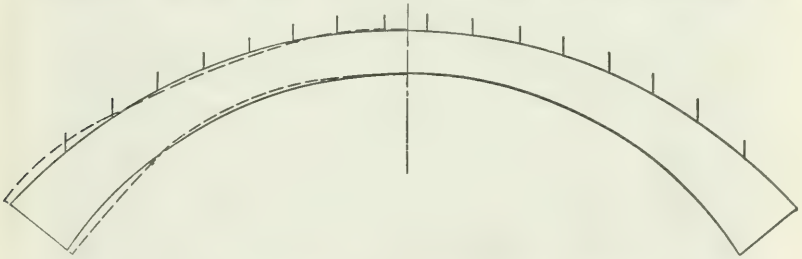


LAYOUT FOR A HINGELESS ARCH OF 500-FOOT SPAN.

FIG. 9.

so close to it as not to affect the economic comparison materially. A small quantity of metal might have been saved by adopting an outline somewhat like that which Gustav Lindenthal, M. Am. Soc. C. E., for motives of esthetics and of suitability to meet existing conditions, but assuredly not for those of economy in weight of metal, used for his two-hinged Hell Gate Arch Bridge. The rib outlines of that long span are, generally speaking, economically appropriate for a hingeless arch, but not for one with hinges at the springing points.

As an approximate test of the correctness of the assumed outline for the 500-ft., hingeless, arch span, find the average of the rib depths at all the panel points and call the result d_a; then compute the average of all the proportioning bending moments at these panel points, and call the result M_a. If M is the proportioning bending moment at any panel point (that is, the larger moment plus one-half of the smaller moment of the opposite sign), find the ratio, r, of $\frac{M}{M_a}$ for each panel point, and multiply it by d_a, then lay out the resulting depth symmetrically above and below the center line of the arch, and join the points thus determined, so as to form new chord center lines. This has been done, as shown in Fig. 10, from which it will be seen that, for five and a half panels on each side of the crown, the

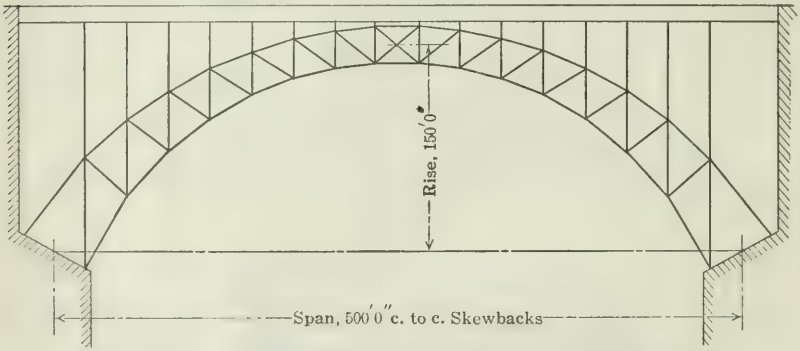


LAYOUT FOR A HINGELESS ARCH OF 500 FEET SPAN, SHOWING MODIFICATION FOR DEPTHS PROPORTIONATE TO ADJUSTED BENDING MOMENTS.
(Solid lines represent original outline as adopted for design, and dotted lines represent modified ideal outline.)
FIG. 10.

curve was drawn just about right, but for the remaining three panels the rib depth should have been greater. As it is not absolutely essential that the rib depth at all points be exactly proportional to the adjusted bending moment, it will suffice to adopt a certain depth at the ends and about half as much at the crown, and run in approximately parabolic curves accordingly.

From Table 6 it seemed likely that, for 500-ft. spans, there is no economy in the hingeless arch as compared with arches having either two or three hinges. However, it was deemed possible that a greater rise might reduce the temperature stresses so as to lower materially the total weight of metal in the hingeless arch. A rough test of this point was made by assuming the rise of the 500-ft. arch to be increased 30 ft., or from 0.24 to 0.30 of the span length, with the result that the

weight of metal was reduced about 7.5% by diminishing the prejudicial effect of the temperature stresses, but was increased about 7% by reason of the greater length of the arch rib, leaving the possibility of some economy by the reduction of stresses from live and dead loads. It was then decided to make an exact determination of the effect of the increased rise, on the layout shown in Fig. 11, even at the expense of 3 days' work for a computer, by making a complete determination of stresses and sections and estimating therefrom the weight of metal. From this it was ascertained that for the arch alone the saving effected amounted to 6%; but, when the proper addition was made for the superimposed columns and their bracing, the gain was exactly exhausted by the increased height from springing point to grade.



LAYOUT FOR A HINGELESS ARCH WITH A RATIO OF RISE TO SPAN OF 0.3

FIG. 11.

It was evident, though, that if the rise of 150 ft. was fixed for the 500-ft. span, the hingeless arch might possibly prove more economical than the three-hinged one; hence, a comparison was made, with the result of a difference of one-half of 1% in favor of the three-hinged type.

The combined highway and electric railway, hingeless arch, with 150 ft. of rise, was also computed, but the saving in the arch alone, as compared with the corresponding span of 120 ft. rise, was only 3%, which was not enough to offset the increase in the weight of spandrel columns and their bracing.

From the four designs (railway and highway) for the 500-ft. span, hingeless arches, there can be drawn a deduction of some value, *viz.*, the manner in which the economics of structures of that type, as compared with three-hinged arches, vary with the ratio of live load plus

impact to total load. In the case of the railroad structure, that ratio is 0.645; and in that of the highway structure it is 0.351. In Fig. 12 the abscissas represent the ratios of live load plus impact to total load, and the ordinates indicate the ratios of weights of metal in hingeless arches to the weights in corresponding three-hinged structures. The right line on the diagram was plotted solely by the two ratios thus determined; and it is assumed to apply properly for all

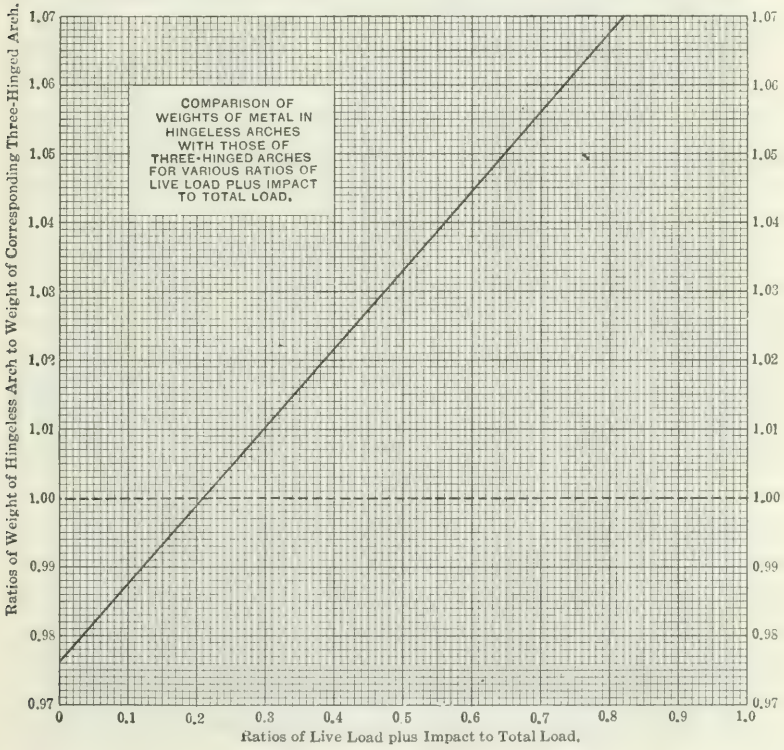


FIG. 12.

spans and loadings, for the following reason: The sole cause of the variation of the economics in the two 500-ft. span, hingeless arches is the factor of the ratio of live load plus impact to total load. Hence, if we know, for several widely divergent span lengths, the ratios of live load plus impact to total load (as can readily be determined for any feasible span by the use of the ten equations previously mentioned, and the numerous diagrams of weights of metal per linear foot of

span)* we can determine, for steam-railway bridges, and for the type of combined highway and electric-railway bridges herein adopted as standard, the economics of hingeless and three-hinged arches. This has been done; and the results have been plotted in Fig. 13, in which the abscissas represent span lengths for arch bridges up to a limit of 1 000 ft., and the ordinates indicate the ratios of weights of metal in

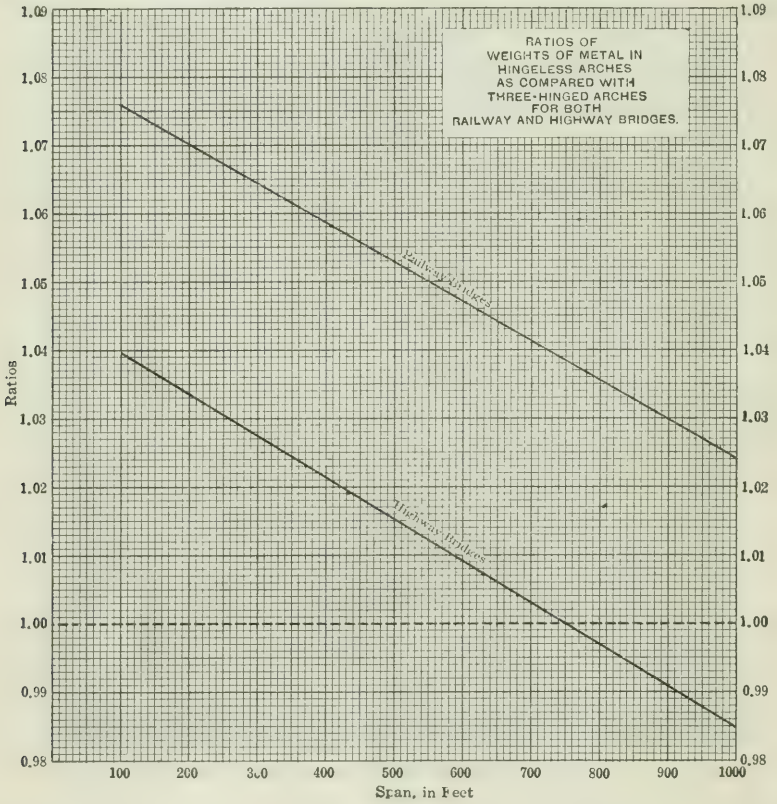


FIG. 13.

hingeless arches as compared with those of the corresponding three-hinged structures. Although the upper line of the diagram was plotted from data obtained for double-track bridges, no error of any magnitude will be made by assuming that it holds good for all strictly railway structures; and the lower line may be used safely for all modern

* Given in Chapter LV of "Bridge Engineering".

highway bridges with paved floors supported by reinforced concrete bases and having concrete sidewalks.

Based on the numerous weight computations made specially for the preparation of this paper, the following formulas for weights of metal in the arches alone, per linear foot of span, in arch bridges of the several types, have been established:

For Three-Hinged Arches.—

$$W_a = (0.000282\ D + 0.000426\ L)\ l \ldots\ldots\ldots\ldots (2)$$

For Two-Hinged Arches.—

$$W_a = (0.000248\ D + 0.000416\ L)\ l \ldots\ldots\ldots\ldots (3)$$

For Combined Two-Hinged and Three-Hinged Arches.—

$$W_a = (0.000282\ D + 0.000416\ L)\ l \ldots\ldots\ldots\ldots (4)$$

For Hingeless Arches.—

$$W_a = (0.000272\ D + 0.000460\ L)\ l \ldots\ldots\ldots\ldots (5)$$

These four formulas are based on the writer's published designing specifications, which treat reversing stresses by adding one-half of the smaller stress to the larger stress and proportioning for the sum; but if the effect of reversion is entirely ignored, as some engineers deem proper, these formulas will reduce to the following:

For Three-Hinged Arches.—

$$W'_a = (0.000292\ D + 0.000396\ L)\ l \ldots\ldots\ldots\ldots (6)$$

For Two-Hinged Arches.—

$$W'_a = (0.000258\ D + 0.000380\ L)\ l \ldots\ldots\ldots\ldots (7)$$

For Combined Two-Hinged and Three-Hinged Arches.—

$$W'_a = (0.000292\ D + 0.000380\ L)\ l \ldots\ldots\ldots\ldots (8)$$

For Hingeless Arches.—

$$W'_a = (0.000270\ D + 0.000398\ L)\ l \ldots\ldots\ldots\ldots (9)$$

In Equations (2) to (9), inclusive, W_a, or W'_a, is the weight of metal, in pounds per linear foot of span, in the arches of the structure; D is the dead load, in pounds per linear foot of span; L is the live load plus impact, in pounds per linear foot, used in making the calculations; and l is the span length, in feet. These eight equations will give fairly

accurate results (slightly on the side of safety) for ordinary conditions which do not vary greatly from the theoretically economic ones.

In computing the value of L for insertion in Equations (2) to (9), inclusive, the equivalent uniform live load and the impact should be determined for the half-span length, as herein previously indicated.

In order to show that the writer's past surmises concerning the economics of the hingeless type of arch were not far wrong, he takes the liberty of quoting the following:*

"It is claimed that hingeless arches require less metal than any other type; but the author, upon general principles only, is inclined to disagree with that opinion. He has never had occasion to design an arch without hinges, but feels quite confident that if he should ever figure one, he would find that a proper provision for excessive temperature stresses, possible large indeterminate stresses, unusual detailing, heavy anchor-bolts, and an inherent sense of fitness in proportioning sections, would so increase the weight of metal as to absorb all of the theoretical economy over the three-hinged arch involved by the smaller average direct stresses as shown on the stress sheets."

Problem No. 6.

From the calculations for the two-hinged and three-hinged types, it was easy to compute the stresses and sections for a combination of the two, making the structures three-hinged for the dead loads and two-hinged for the live loads. The results showed, as was anticipated by *a priori* reasoning, that the combination produces weights of metal intermediate between those of the two types previously computed. The exact results are given in Table 7.

TABLE 7.

Character of structure.	Weight of metal, in pounds per linear foot per side.	Percentage compared with three-hinged structure.
Railway arch	2 108	97
Highway arch	1 212	99

From Tables 7 and 6, it will be seen that there is an increase of weight of 2% for the railway structure and of 7% for the highway structure in the combination, as compared with the two-hinged type; but the resultant excess of cost will generally be considered of small importance in comparison with the value of the certainty of the dead-

* From p. 621 of "Bridge Engineering."

load distribution effected by the hinging, combined with a reduction in the difficulty and expense of erection.

As compared with the three-hinged arch, the combination type affords greater rigidity under moving load—an especially important feature in railroad bridges. Again, the erection problems are less difficult, and therefore less expensive, for three-hinged than for two-hinged arch structures.

Problem No. 7.

The solution of the problem of determining the economics of the cantilever-arch bridge was accomplished thus:

First.—A design was made for a three-hinged, spandrel-braced, double-track railway, arch bridge of 500-ft. span and of approximately economic rise; and the weight of metal for the arches thereof was then computed.

Second.—Simple-truss flanking spans, each 292 ft. long, were added, resting at their outer ends on rollers, and attached at their inner ends to the vertical end-posts of the arches; and the weight of metal for these flanking spans was calculated.

Third.—It was assumed that the arches were cantilevered out for three of the seven panels, and that the remaining four panels formed the suspended span; then the weights of both the arch span and the flanking spans were computed. This layout was deemed in advance by the writer to be the most economic one possible.

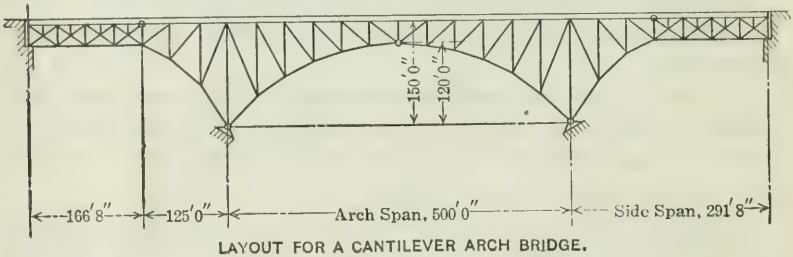
Fourth.—Next, the number of panels in each cantilever arm was reduced to two; and, as before, the weights of metal in all spans were calculated.

Fifth.—Finally, the number of panels per cantilever arm was increased to four, and the work of computation was repeated.

As surmised, there was quite an economy of metal by cantilevering, and the three-panel, cantilever arms proved to be the most economical, the saving of metal in the arches and trusses combined being about 11 per cent. There was a small increase, instead of a reduction, of weight of metal in the arch, but the economy in the flanking spans was quite marked. The best lengths of cantilever arm and suspended span were, respectively, about 40 and 60% of the total length of one flanking span. The layouts for two of these computed structures are shown in Figs. 14 and 15, the former being for the cantilever arch, and the latter for an ordinary arch with flanking simple-truss spans.

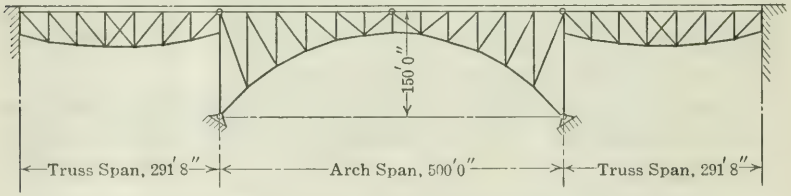
It was thought possible that a different length of flanking span would entail different results; hence, the length was reduced to 208 ft., and the calculations were repeated. As in the last case, there was a saving in the combined weights of arches and trusses, but not so pronounced an economy as before; however, the economic ratios for cantilever arm and suspended span to total length of flanking span remained unchanged, being, respectively, about 0.4 and 0.6.

It was for convenience of computation that the arches in these comparisons were made three-hinged; but, it is evident, after giving the question a little thought, that the conclusions reached would have been practically the same had the arches been two-hinged. Hingeless arches, of course, are not suitable for the cantilevering of flanking spans.



LAYOUT FOR A CANTILEVER ARCH BRIDGE.

FIG. 14.



LAYOUT FOR AN ARCH BRIDGE, WITH FLANKING SIMPLE-TRUSS SPANS.

FIG. 15.

PROBLEM NO. 8.

A few years ago the writer made an attempt to obtain, by correspondence with a large number of bridge engineers, some reliable data relating to the ratios of weights of metal in arch bridges and truss bridges of the same span length and the same live-load carrying capacity; but he was unsuccessful, as nothing reliable on the subject was known to the Engineering Profession. However, from the numerous calculations made for this paper, combined with weights of simple-truss spans previously published by the writer, there has been prepared the diagram, Fig. 16.

The right lines plotted thereon coincide with those recorded by the following equations:

For Steam-Railway Structures.—

$$P = 106 - 0.08\ S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

For Electric-Railway Structures.—

$$P = 93 - 0.07\ S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

For Highway Structures.—

$$P = 80 - 0.056\ S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

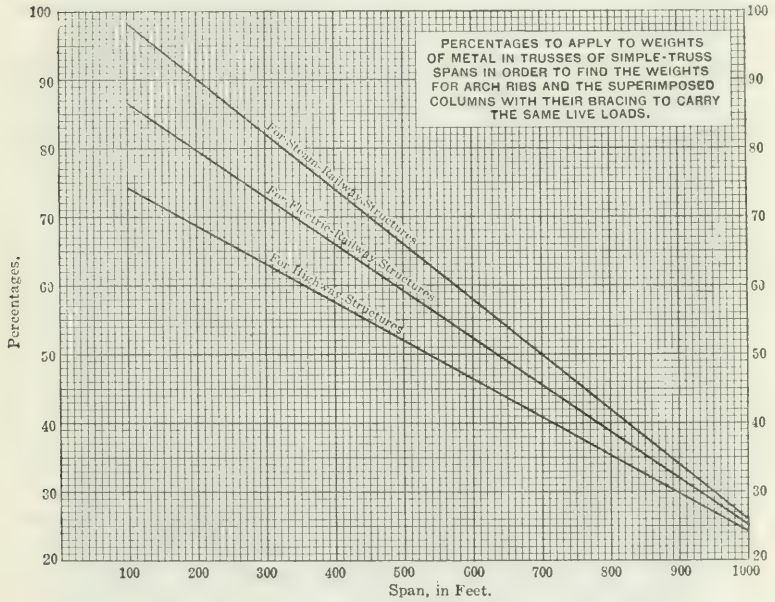


FIG. 16.

In these equations, S is the span-length, in feet; and P is the percentage to apply to the weight of metal in the trusses of any simple-truss bridge, in order to ascertain the weights of corresponding arches and the superimposed columns with their bracing. It must not be forgotten that the superior limit of S in these two equations is about 1 000 ft., which is as far as the recorded weights of simple-truss spans are carried, and that the inferior limit is 100 ft. The two exterior lines of the diagram were computed, but the middle one was interpolated by judgment. None of them can be considered as strictly

accurate for all cases, because of the influence of varying conditions precedent.

From Fig. 16 and Equations (10) and (12) it is evident that the arch is more economical for combined highway and electric-railway bridges than for steam-railway structures. This is because of the

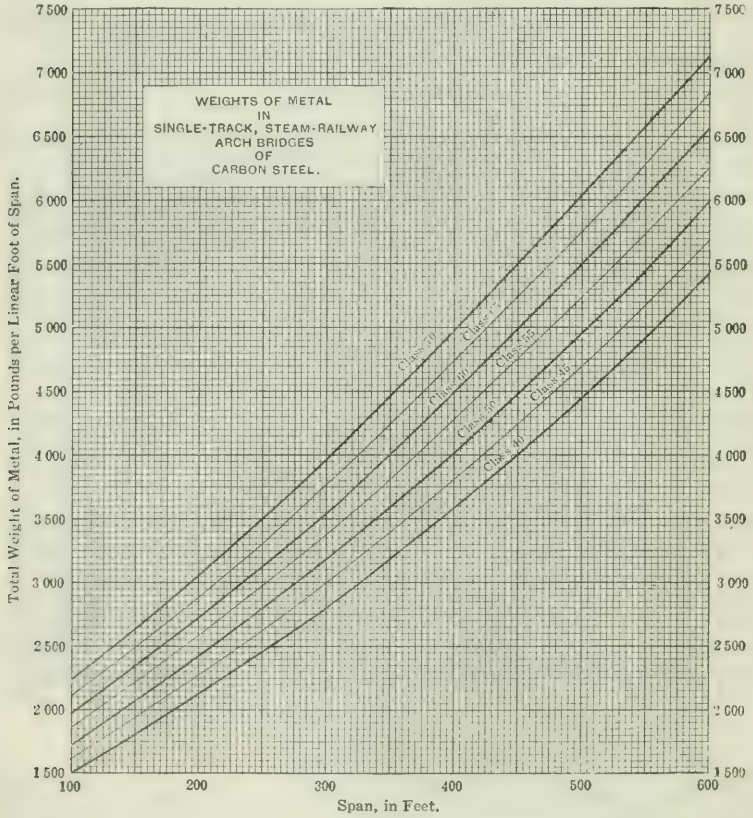


FIG. 17.

smaller ratio of live load plus impact to total load in the former. The larger the dead load of the flooring and floor system, the more advantageous is it for the arch structure in comparison with the truss bridge. In the double-track, steam-railway bridges of this investigation, the floor weight per linear foot of span is about 2 200 lb., and in the 60-ft. wide, highway and electric-railway bridge thereof, with its paved floor,

reinforced concrete base, and reinforced concrete sidewalks, it is about 6 400 lb. In Fig. 16 the converging of the three right lines as the span length increases is due to the augmenting influence of the dead load.

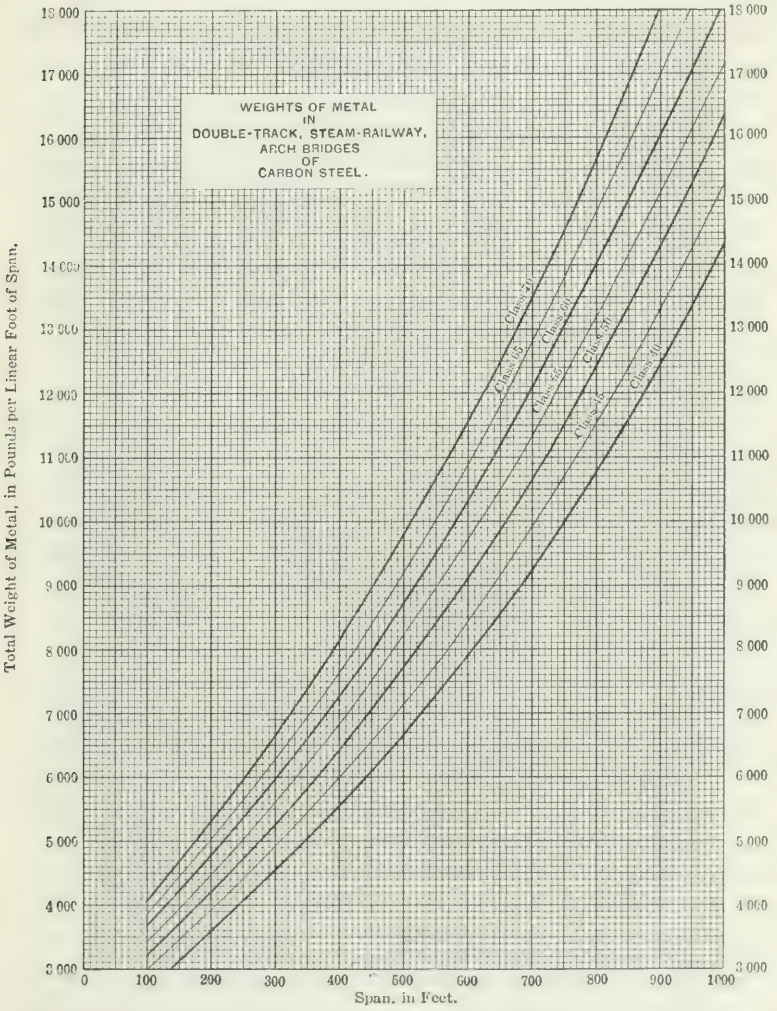


FIG. 18.

No comparison has been made between the weights of solid-rib arches and those of the corresponding truss spans, because such a comparison would seldom be of any use.

By using the percentages shown in Fig. 16 and certain diagrams of weights of metal previously published by the writer, there have been prepared the diagrams of total weights of metal per linear foot of span for three-hinged, braced-rib, arch bridges, shown in Figs. 17, 18, and 19. These are, respectively, for single-track, steam-railway bridges; double-track, steam-railway bridges; and combined highway and electric-

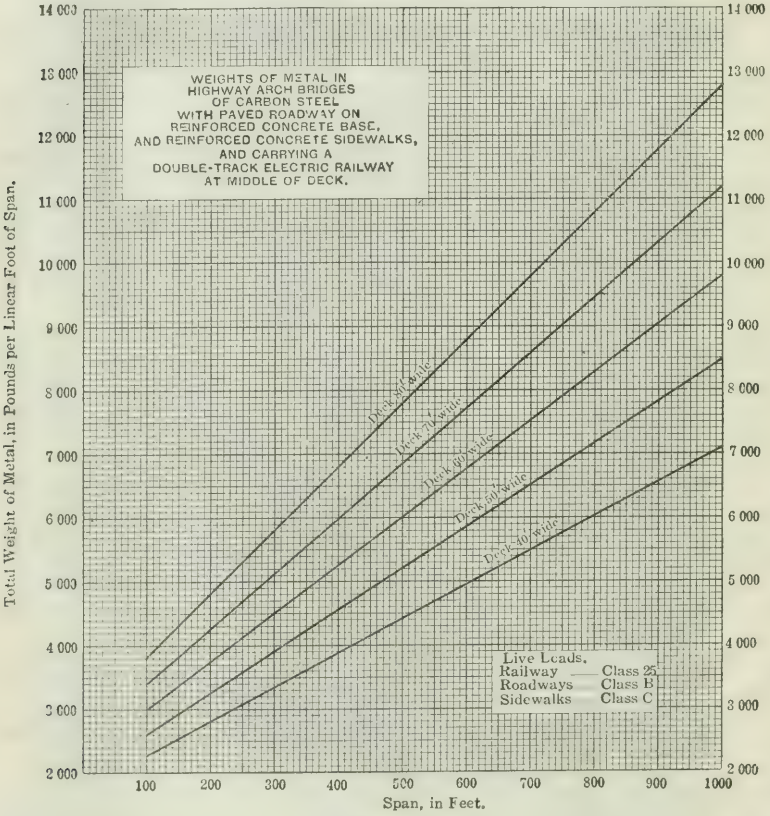


FIG. 19.

railway bridges, with paved floor, reinforced concrete base, and reinforced concrete sidewalks. In preparing these diagrams, due account has been taken of the splaying of the columns in the bents carrying the floor. If these diagrams are to be used for spandrel-braced arches by making a small percentage reduction or addition, based on the deduc-

tions in Problem No. 4, it will be necessary to multiply the result thus found by the secant of the angle which the splayed spandrels make with the vertical. In order to determine the proper splay for any case, one should remember that the width of structure at the springing should never be less than 30% of the vertical distance from the springing point to grade, and, preferably, not less than 35% thereof.

For weights of metal in multiple-track, arch structures, no error of any importance will be made by assuming them to be directly proportional to the number of tracks (using the double-track records of

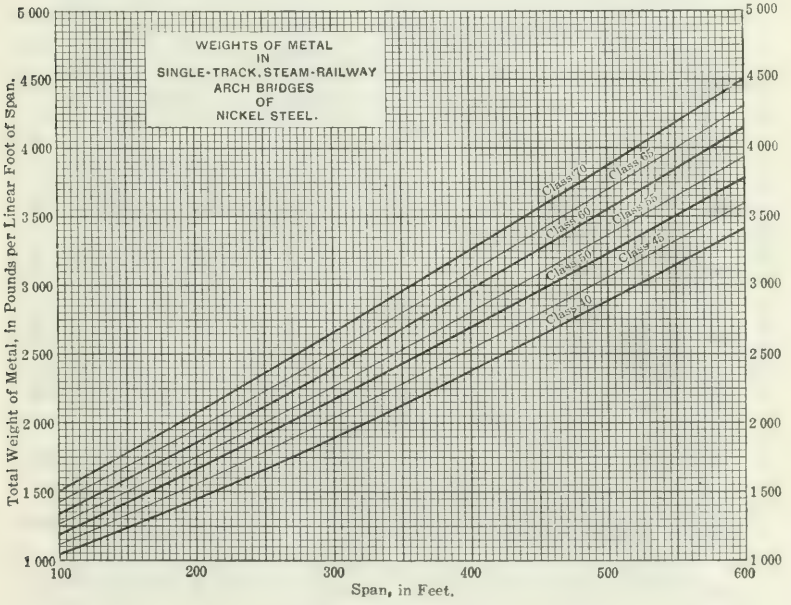


FIG. 20.

Fig. 18 as a basis); and for strictly highway arch structures of similar construction throughout, by taking them proportional to the out-to-out width of the deck. In Fig. 19 this proportionality does not hold, because the structures therein diagrammed carry at mid-deck a double-track, electric railway.

It seems hardly necessary to state that for hingeless, two-hinged, and combined three-hinged and two-hinged arches, the total weights of metal per linear foot of span may be readily obtained by applying to the weights, found from Figs. 17, 18, and 19, the small percentage

corrections for arch weights hereinbefore determined in the discussion of the economics of the different types, using Equation (2) for finding these arch weights.

It may be of interest to some engineers to know the weights of metal in steam-railway, arch bridges built of nickel steel; consequently, the diagrams, Figs. 20 and 21, have been prepared for single-track and double-track, arch structures, respectively. It seems hardly worth

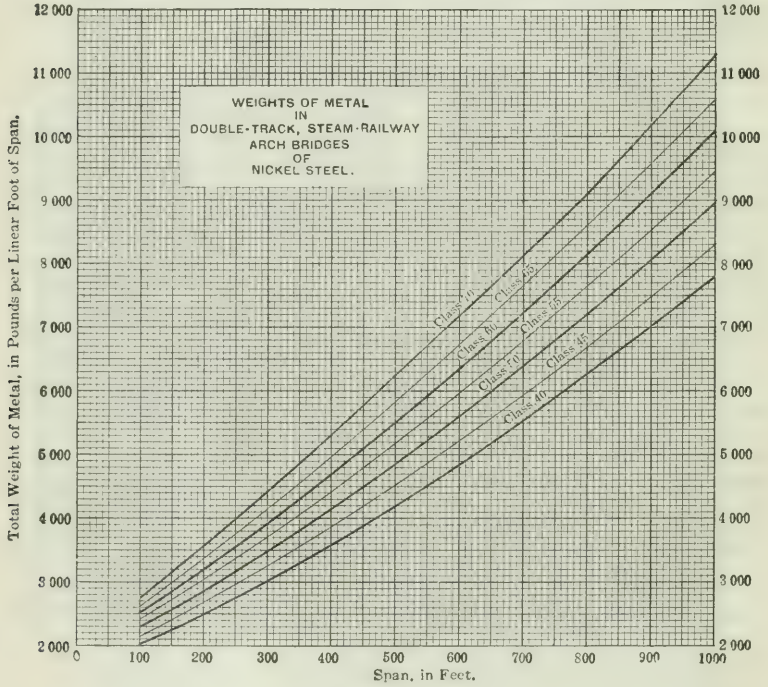


FIG. 21.

while, however, to give diagrams for highway, arch bridges of nickel steel, because of the uncertainty as to the extent to which the alloy would be used in any particular structure. For bridges of comparatively short span lengths, it might be economical to use it in the arches only; in long spans, in which the importance of reducing the dead load is paramount, it would pay to use it in the floor system also; but in very few cases is it ever advisable to adopt the alloy for the lateral system, on account of the fact that the proper minima sections of

carbon steel generally provide an excess of strength for the computed wind stresses.

From the formulas herein given for weights of arch ribs, some interesting deductions may be drawn. For instance, in Equation (2), *viz.*,

$$W_a = (0.000282\ D + 0.000426\ L)\ l,$$

the dead load, D, is composed of the rib weight, W_a, plus the weight

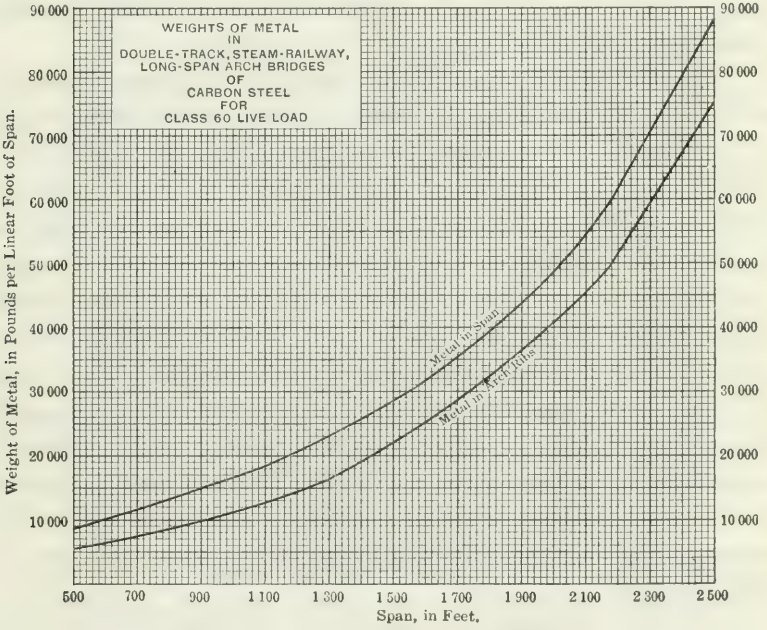


FIG. 22.

of floor, columns, lateral system, etc., all of which may be grouped under the symbol, W', making the equation

$$W_a = (0.000282\ W_a + 0.000282\ W' + 0.000426\ L)\ l.$$

Solving this gives:

$$W_a = \frac{(0.000282\ W' + 0.000426\ L)\ l}{1 - 0.000282\ l}.$$

In order that W_a may be infinitely great, the divisor of the second term must be equal to zero, or

$$l = \frac{1}{0.000282} = 3\,540 \text{ ft.}$$

This is the theoretical limiting span for three-hinged arches of carbon steel, or the span at which such an arch could carry nothing but its own weight without being over-stressed. It will be noted that this limiting length is the reciprocal of the dead-load coefficient in Equation (2), and that a vertical line on the diagram of arch rib weights per linear foot of span, drawn through the abscissa point which represents this value of l, will be asymptotic to the weight curve. Such a weight curve for long spans of steam-railway bridges is partly shown in Fig. 22.

From this curve it is possible to determine the economic or practicable limit for arch spans, by assuming, as the writer did years ago, in his economic investigations for cantilevers, that the said limit is reached when it takes $4\frac{1}{2}$ lb. of metal to carry 1 lb. of live load. On that basis, and assuming that the equivalent uniform live load per linear foot for a long span is equal to the car load per linear foot, the limiting weight of metal per foot for carbon-steel, double-track, steam-railway bridges of Class 60 would be $4.5 \times 12\,000 = 54\,000$ lb. Referring to Fig. 22, it is found that the span for that weight is nearly 2 100 ft.; consequently, generally speaking, it may be stated that, for steam-railway, arch bridges of carbon steel, the limiting length of span is about 2 000 ft., or the same as the limiting length of main opening in cantilever bridges built of the same material. Judging by analogy, the corresponding practicable limiting length of nickel-steel arch spans is about 2 600 ft.

The various formulas and diagrams in this paper are to be considered as merely approximate; and though they are sufficientiy accurate for preliminary estimates and for obtaining trial dead loads, they should not be used by contractors in tendering on work. The reason for this uncertainty is that the varying physical conditions at different crossings affect the arch layouts to such an extent as materially to influence the weight of metal required. As the formulas were based on economic functions, the weights given by their use might very properly be considered as the minima possible; and any uneconomic conditions which may exist will involve an increase thereof, the amount being a matter to be determined by the computer's judgment.

Résumé of Results.

The following is a condensed statement of the principal findings from all the preceding investigations, recorded in the previously indicated order of the problems set for solution:

First.—For three-hinged arches with the grade line approximately tangent to the top chord of the arch at the crown, the average economic ratios of rise to span length are as follows:

Solid-rib structures	0.2
Braced-rib structures	0.225
Spandrel-braced structures (with hinge above)	0.25

These values may be either increased or decreased by 0.025 without making any material difference in the economics.

For three-hinged half-through, arch bridges, the average economic ratios of rise to span-length are as follows:

Solid-rib structures	0.225
Braced-rib structures	0.3

For three-hinged, high-deck, arch bridges, they are as follows:

Solid-rib structures	0.25 to 0.28
Braced-rib structures	0.33 to 0.38

For two-hinged arches and combined two-hinged and three-hinged arches, the economic ratios of rise to span will be practically the same as for three-hinged structures.

For the hingeless arch, a somewhat greater ratio of rise to span than that for the three-hinged arch is economical. The single test of this made for the 500-ft. span indicates that the best ratio is about 0.28 with low-grade deck, 0.33 for half-through arches, and 0.38 with high-grade deck.

Second.—In respect to solid-rib arches, the question of economic depth of rib does not arise; for the depth should always be made as great as a proper consideration of the section for resisting compression will permit. For braced-rib, three-hinged arches the economic rib-depth is about 7.5% of the span length, but quite a variation from

that percentage may be made without materially increasing the weight of the metal.* The economic average depths for hingeless and two-hinged arches will be about the same as the economic depths for three-hinged arches. The best form for a two-hinged arch is the crescent shape, but there is no great economy in this over the rib with uniform depth. The best shape for the hingeless arch is secured by making the depth at the crown about one-half of that at the springing points. Such a layout, as shown in Fig. 11, tends not only toward uniformity of chord section, but also toward esthetics.

Third.—It was found for all cases that the most economic location for the crown-hinge in a spandrel-braced arch is in the top chord. In braced-rib and solid-rib arches, however, the hinge should always be placed at mid-depth, so as to distribute properly the thrust over the two chords.

Fourth.—Comparing the economics of solid-rib, braced-rib, and spandrel-braced arches, it was found that the first mentioned is always considerably heavier than either of the others; and, under normal conditions of the metal market, it is also more expensive. The spandrel-braced arch requires for long spans a little more metal than the braced-rib arch; but, in case of cantilevering, this would generally be offset by the extra quantity of erection metal needed for the braced-rib structure. In short spans, the spandrel-braced arch is a trifle the lighter.

Fifth.—Comparing three-hinged, two-hinged, and hingeless arches, it was found that the three-hinged is a little heavier than the two-hinged, but nearly always lighter than the hingeless. The variations in weight among the three types, however, are never great.

Sixth.—The combination of three hinges for dead load and two hinges for live load produces very little saving over the three-hinged type; but it adds materially to the rigidity. In the writer's opinion, the combined type is preferable to any of the others.

Seventh.—When an arch is flanked by other spans than arches, there is generally quite a little economy involved by cantilevering the ends of the arch and shortening the lengths of the simple spans. The best proportions for lengths of cantilever arm and suspended span to total length of flanking span are, respectively, 0.4 and 0.6.

* For a modification of this see the résumé of the discussions.

Eighth.—The ratios of weights of arches to the weights of the corresponding simple spans for both railway and highway bridges have been determined and plotted. As was anticipated, the arch usually effects a greater relative economy in highway structures than in railway structures of the same span length; and, the longer the span the greater always is the proportionate saving of metal.

As a conclusion to this paper, the writer begs to quote from one of his previous publications as follows:

"In dealing with the comparative economics of arches and simple trusses, it must not be forgotten that there are other factors than mere weight of metal involved; for the pound price of the manufactured material is generally somewhat greater for the former, and sometimes the cost of erection also is larger. Again, the comparison of the costs of arch superstructures and truss superstructures alone is not of much importance; for an economic investigation, to be of any value, must include both substructure and superstructure; and the costs of the former are likely to be very different in arch designs and simple-truss designs for any crossing".

The thanks of the writer are due to D. B. Steinman, Assoc. M. Am. Soc. C. E.,* one of the associate engineers of his firm, for expert assistance in making certain graphic calculations by which a number of important points of economics determined in this paper were solved.

* Now M. Am. Soc. C. E.

DISCUSSION

Mr. Blackwell.

PAUL A. BLACKWELL,* ASSOC. M. AM. SOC. C. E.† (by letter).—The author has again enriched engineering literature by his latest contribution, "The Economics of Steel Arch Bridges." Any one who has had to do with arch work knows the difficulty of getting working data from the mass of mathematical information on this subject. This has resulted in the decision of the average engineer to steer clear of arches of all descriptions, because they are difficult to calculate; and that there ever were any economical considerations applying to structures of this type, probably never entered his mind.

It is true that arches are generally built as monuments, or for advertising purposes, and are chosen for these purposes because they are easily adapted to esthetic treatment; and, in view of their use only as monuments, the question of economy has hardly been considered.

Several years ago the writer, having in view something of what the author has brought out, endeavored to make a comparison between the economical advantages of the generally accepted three types of arch. Reducing the results that were found to the same classification as given in this paper, the writer will comment as follows:

First.—In reference to the economic ratio of rise to span, it seems to be the consensus of opinion that the correct ratio is about 1:5. This agrees with the ratios given, although this, like everything else that has to do with bridge work, must depend on what can be done, and not always on what should be done; and, of course, as the type of arch chosen must be the best suited for the place, the ratio of rise to span is not dependent on the type.

Second.—The writer takes exception to the following statement: "As far as solid-rib arches are concerned, the question of the economic depth of the rib does not arise." As so many things enter into the problem—one of which is shipping—the available depth is often determined. This may not be the theoretical economical, or least-weight, depth; but, when ease of manufacture, obtaining material, and ease of erection are considered in their relative importance as economical factors, the limit of the solid-web arch is reached with about a 10-ft. web, which would correspond to a probable maximum span of 400 ft. In nearly all existing arches the general practice has been to make the depth of ribs about one-twenty-fifth to one-fiftieth of the span. The writer generally uses one-fortieth, about one-third of what the author finds as the economical depth. A depth of 60 ft. for an 800-ft. arch rib would require members of the webbing about 80 ft. long. These members carry comparatively small stresses, and, in order to be so long, would have to be very large to have sufficient stiffness, and would probably be as large as the chords themselves. This would

* Roanoke, Va.

† Now M. Am. Soc. C. E.

not be economical; besides, the appearance of the arch is improved by reducing the depth of the web. Varying the shape of the hingeless arch (widest at the ends) adds to the appearance and results in economy of material. Mr. Blackwell.

Third.—The writer does not believe that there is any economy in putting the crown-hinge in the top chord. Generally, a neater detail, and a more easily erected and better looking structure results from placing the crown-hinge in the bottom chord (for spandrel-braced arches). In all other cases the crown-hinge should be placed as the author recommends.

Fourth.—The writer believes the spandrel-braced, two-hinged arch, of moderate span, to be the most satisfactory structure for railway traffic, when an arch is necessary or desired; but, for highway structures, should the condition allow, an arch-rib structure is better.

Fifth.—The writer has found that the three-hinged arch is the heaviest type; the two-hinged and hingeless are of about the same weight. The hingeless arch is the stiffest, but, in order to make the ends truly fixed, there is no economy in weight over the two-hinged arch.

Sixth.—The accepted modern method is to calculate the arch as three-hinged for dead load and erection, and then rivet up in the field, making it two-hinged for live load and temperature. It relieves the arch of much secondary stress, and causes the joints to come to their bearing. It is much the better way.

Seventh.—The arch with cantilever spans makes a pleasing structure. The economy, however, is questionable, considering the lack of rigidity of all cantilevers, and the reduction in the weight of the arch proper is slight; most of the saving in weight is due to the change of length of the side spans. It is always more economical to carry loads in tension or compression than in bending.

Eighth.—The field of the arch, for purely economical reasons, lies somewhere between 500 and 1 000 ft. The writer's experience in this line has been with small arches (250 ft.) which weigh practically the same as a simple span of the same length.

In conclusion, the writer fully realizes the vast amount of labor involved in getting these results, and the author is to be congratulated on the orderly manner in which he has presented his findings.

T. Kennard Thomson,* M. Am. Soc. C. E.—The author deserves the thanks of the Profession for the immense labor of love he has performed in making these calculations and giving engineers the benefit of the results. Mr. Thomson.

No one recognizes better than Dr. Waddell the fact that environment and special features of topography and scenery will often control the ratio of rise to span of the arch to be built, rather than the quantity of steel in the structure.

* New York City.

Mr. Thomson.

For instance, the speaker's design, Fig. 23, for a three-hinged steel arch bridge encased in concrete, to cross the Niagara River just below the Falls, has a rise of only 0.13, as compared with the author's figures of 0.20 to 0.38. In other words, the rise of this bridge will only be about 110 ft. for a span of 840 ft.

If the design were changed to give a rise of 0.25, then its height would be 210 ft., or 100 ft. higher; and in that case the floor would have to be raised 100 ft., if the deck-bridge design is to be retained, or the sides or trusses would have to extend 100 ft. or more above the floor. Neither of these changes should be allowed, in this case, as the bridge should not be so unduly conspicuous—especially in the neighborhood of the Falls; and, besides, the excess of concrete used would probably counterbalance the saving in the steel. The design as it stands is arranged to give the least possible obstruction to the view of the Falls, and has the appearance of naturally filling its proper location between the cliffs. In other words, any well-designed engineering structure will look as if it were put there by Nature, and, like a well-dressed man, will not cause a question regarding its *raison d'être.*

While speaking on the subject of economy of design the opportunity might be taken to refer to a detail of economy which applies to nearly all steel bridges, whether arches or not, but which, unfortunately, often seems to be neglected, although it is very important. It is the comparison between the efficient and excess weight of steel in a bridge. By the efficient weight is meant the weight of the plates or shapes which go to make up the necessary section to withstand the stresses. By the excess weight is meant all other steel, such as pin-plates, splice-plates, lattice-bars, pins, bolts, rivets, etc. Naturally, economy requires that the excess weights be kept as small as good design will permit.

On a small bridge, of course, there is not so much leeway as on a large one; but, for very large members, within which a painter can work, a box or circular column or strut would often be more economical than the usual web systems which are held together by lattice-bars, diaphragms, etc. When the bridge is built under a "pound price", however, there is a temptation to use the heavier sections, especially as they can be handled more easily in the shop.

The first essential in designing a column would be to make it equally strong in any direction; but this is difficult to accomplish economically when it is built up of several heavy ribs connected by latticing, etc. An economical light column can be secured by riveting an I-beam between two channels, giving a section having very little excess. This permits painting, which a small box-column does not. Where the sections are large enough to permit a man to pass through, a box or circular section would be more easily protected against rust,

Fig. 23.—Proposed Bridge over the Niagara River near the Falls. Three-hinged Steel Arch, Encased in Concrete. Main Span, 840 Feet; Height Above River, 143 Feet.

and the saving in weight should overbalance the increase in cost of labor. Mr. Thomson.

If this rule—equal strength in any direction—had been followed in the design of the first Quebec Bridge, the disaster at that site would probably never have occurred.

The speaker knows that it would be presumptuous to try to teach Dr. Waddell anything about bridge design, but he trusts that he will be pardoned for inserting these self-evident, but often needed, remarks, for the benefit of students of bridge building.

Charles Evan Fowler,* M. Am. Soc. C. E.—The great amount of detail work involved in such a paper as this is not fully realized unless one attempts a thorough investigation of the subject. The Profession is much indebted to the author for making this systematic study of arches and for recording the results of such an extensive series of calculations. An independent examination of the subject reveals almost as wide a divergence from the author's views as there are modifications of each class of structure. Mr. Fowler.

The speaker has always believed the arch to be a splendid type of structure to "fill up a hole" or fit into a given gorge, with but little latitude for deck spans to influence true economy, if one adheres to esthetics in the best sense. This seems to be borne out by a study of the one hundred arches in Table 8 (Plate I), ranging in span from slightly less than 200 to nearly 3 000 ft. That is, such is the general conclusion with regard to deck spans, but, of course, through or half-through arches can always be designed with a real regard for economy.

The fundamental idea to be borne in mind, however, is that, even though many deck arches cannot be designed with the most economic depth, and, consequently, are not economical structures when considered alone, the total cost of the structure, including foundations, will usually be much less than that of a bridge of any other class, where there are rocky walls or a rock foundation to take up the thrust, and with slight expense for preparing the skewbacks.

Although only one hundred arches of more than 200 ft. span have been found and listed, twenty-four of which have not been actually built, arches of shorter span are probably more numerous than bridges of any other type, except perhaps simple trusses; and if stone, concrete, and reinforced concrete arch bridges are included, the number is legion. The case under consideration has to do with steel or framed arches, and the study of the list referred to has been undertaken on the assumption that each designer has had a reasonable regard for all theoretical and practical considerations, and has selected the most economical type of arch and the best theoretical proportions for it, when not hampered by unalterable conditions. The more the various structures and designs are studied, the more certain is it that there

* New York City.

Mr. Fowler.

are many flagrant departures from economy, especially in the case of the shorter spans which do not warrant a great expenditure for investigating independently the economic possibilities. The speaker has always endeavored in special cases to make such investigations, but has always believed that the economic rise was practically equivalent to the depth of a simple truss span. In other words, given a certain type of arch, the rise must be selected as nearly as possible in the same way that the depth of a truss is found, that is, deeper for the heavier and dynamic loadings. Where one-sixth rise might prove enough for producing a stiff structure for the light loading of a highway arch bridge, it would need to be increased to one-fifth, or perhaps to one-fourth, or more, for heavier loading and for railways, using the engineer's judgment for preliminary layouts to form the basis for investigations. This, therefore, does not agree fully with the author's findings. Judgment alone would have indicated that the Washington Bridge over the Harlem should have had a greater rise than 18% if in any way possible, or, the girders should have been deeper, and thus have avoided in large measure the vibration or bouncing of the structure caused by the live loads. The rise necessary to overcome this would have been much greater than the economical rise found by the author, or probably 25% of the span, or greater. The same unpleasant feature occurred in the speaker's Knoxville arched cantilever, and, in both cases, it is due largely to the inertia of a heavy paved floor, a condition which cannot be entirely overcome. The ultimate limit of economy should not be considered as the only thing to be attained; the aim should be to design a structure that shall be stiffer and therefore more lasting, without, of course, any undue increase in cost. The Alexander III Bridge, in Paris, is an even more flagrant example of a too shallow arch, and, with a rise of only one-seventeenth, the vibration, of course, is as predominant as it is lacking in arches of possibly too great rise, like the speaker's White Pass, Alaska, arch, with a rise 37.5% of the span, or the St. Croix, Minn., arch spans, with 35.5% rise.

The discussion of the subject can be taken up more understandingly by considering at this point the various types of arches that have been built (or designed), as shown by the list (Table 8, Plate I) prepared by the speaker. This list is believed to be several times more extensive than any previously compiled. The data have been checked from the original sources, as far as can be found in the Engineering Societies Library, and many errors have been corrected in data which have been in current use for some years. The various types were found to be as follows:

(*a*).—Arch ribs with no hinges, plate girders;
(*b*).—Arch ribs with no hinges, lattice ribs;
(*c*).—Arch ribs with no hinges, trussed ribs;

TABLE 8.—ONE HUNDRED STEEL ARCH BRIDGES, FROM 200 TO 3 000 FEET SPAN.

No.	Location.	Span, in feet.	Rise, in feet.	L ÷ R Proportion.	R ÷ L Fer-centage.	Weight per cubic ft.	Rib Depth, in Feet. Center	Springing	Per-centage	Kind of traffic.	Class.	Remarks.	No. Hinges.	Engineer.	Date built or designed.	Reference.
1	Hudson River, New York City†	2 850.0	440.0	6.5	15.5	30.0	100.0	400.0	3.5	H.T.	Lat. Rib.	Longest arch ever designed. Bold erection plan.	2	Max am Ende.	1896	*The Engineer*, May, 1899.
2	Quebec, Canada§	840.0	154.0	11.0	18.1	32.0	91.0	43.0	10.8	H.T.	Spand. Arc.	Vonsoir arch. Thrust resisted by concrete struts.	2	Washington.	1916	*Engineering News*, May, 1910.
3	York, Scotland	630.0	390.0	6.0	30.0	30.0	71.0		4.8	H.T.	Par. Lat. Rib.	Comparison of 3-hinged parabolic rib with cantilever.	3	Max am Ende.	1893	*The Engineer*, May, 1893.
4	" "	540.0	324.0	6.0	34.0	30.0	59.0	50.0	1.4	H.T.	Lat. Rib.	Very shallow, no-hinge lattice rib with internal ties.	0	" " "	1896	*Engineering*, Feb., 1896.
5	Quebec, Canada§	1 600.0	320.0	9.0	30.0	30.0	89.0		4.8	H.T.	Par. Lat. Rib.	Investigation of 3-hinged parabolic rib for this site.	3	" " "	1898	*The Engineer*, May, 1898.
6	Sydney, New South Wales§	1 500.0	297.0	6.0	30.0	30.0	89.0		4.8	H.T.	Par. Lat. Rib.	Investigation of 3-hinged parabolic rib. Valuable.	3	" " "	1898	" " "
7	Bosphorus, Turkey§	1 940.0	340.0	5.8	19.8	30.0	58.0		4.8	H.T.	Par. Lat. Rib.	Comparison of 3-hinged parabolic rib with other bridges.	3	" " "	1898	" " "
8	Thames, London, England§	1 150.0	270.0	4.6	23.5	30.0	59.0		5.1	H.T.	Par. Lat. Rib.	3-hinged parabolic rib. Notes scheme below Tower Bridge.	3	" " "	1896	*The Engineer*, March, 1896.
9	Hoi Gate, New York City§	977.5	220.0	4.4	22.5	30.0	40.2	160.0	4.1	H.T.	Truss Rib.	Longest arch built. Economical and artistic	2	Lindenthal.	1916	Proc., Am. Soc. C. E., Oct., 1917.
10	" " §	977.5	320.0	4.0	32.7	30.0	60.0	0.1	1.9	D.	Lat. Rib.	Study of 3-hinged crescent arch for No. 9 location.	3	" " "	1915	Proc., Am. Soc. C. E., Oct., 1917.
11	Kentucky River§	884.0	197.5	5.8	19.8	29.3	49.8		5.7	D.	Hy.	Lat. Rib.	2	"	1917	*Engineering Record*, Nov. 29, 1919.
12	Harlem River, New York City§	880.0	180.0	6.6	15.8	35.0	28.0	109.0	8.8	D.	Hy.	Spand. Br.	2	Hedge.	1917	*Engineering Record*, Nov. 29, 1919.
13	Niagara, N. Y	840.0	150.0	5.6	17.8	30.0	28.0		3.3	D.	Hy.	Lat. Rib.	2	R. B. Buck.	1897	Waddell, Br. Engr.
14	" " §	840.0	110.0	7.6	13.1	62.5	90.0	80.0	9.4	D.	Hy.	Lat. Rib.	2	Thomson.	1918	*Engineering Record*, 1908-2.
15	Vaur, France	752.0	176.0	4.3	24.0	30.0	11.0	69.0	1.4	H.T.	Par. Lat. Rib.	Longest cantilever arch ever constructed.	3	Boulin.	1896	*Engineering News*, 1903-B.
16	Viaur, France	628.0	150.0	4.3	23.8	30.0	4.4	85.2	0.7	H.T.	Par. Lat. Rib.	Novel combination 3-hinge and 2-hinge. Erection striking	3	Lebert.	1918	*La Génie Civil*, Nov., 1918.
17	Rhine, Germany	614.0	97.0	6.3	15.8	30.0	15.7	54.8	8.9	D.	Hy.	Braced rib with tension tie. Very artistic.	2	Krohn.	1919	*Engineering News*, 1904-1.
18	Ingolstadt, Germany	594.7	90.5	6.6	15.2	30.0	15.7	31.6	5.7	V.	Truss rib.	Similar in design to No. 17 at Bonn.	2	"	1920	*Engineering News*, 1903-2.
19	Neuhof, Austria	592.0	102.0	4.1	17.2	30.0	15.0		2.5	H.T.	Hy.	Lohse and Ingold 3-hinge lattice rib built.	3	Scowrtise.	1915	*Engineering Record*, Nov. 11, 1916.
20	Caveland, Ohio	591.0	144.0	4.1	24.3	30.0	60.0	91.3	3.4	9 D.	Hy.	Truss Rib.	3	Feirano.	1915	*Engineering News*, June 28, 1916.
21	Lute I, Douro, Portugal	568.0	148.4	4.0	26.0	30.0	26.2	66.0	4.7	D.	Hy.	Carries two roadways, at low and high level.	2	Seurig.	1886	Mebrieau.
22	" " §	568.0	142.0	4.0	25.7	30.0	Av. 29.0	Av. 69.0	4.6	2 D.	Hy.	Comparative design, from original competition.	0	Hardyside.	1889	*Engineering*, Dec. 24, 1890.
23	" " "	568.0	155.0	3.7	27.4	29.8	18.0		3.2	D.	Hy.	Comparative design, 3-hinge lattice rib.	3	Eiffel.	1890	" " "
24	" " "	568.0	155.0	3.7	27.4	29.8	18.0		3.2	D.	Hy.	Comparative design, 2-hinge lattice rib.	2	Cail.	1890	" " "
25	" " "	568.0	151.7	3.8	26.7	30.0	28.6	89.3	4.6	9 D.	Hy.	Comparative design, 0-hinge lattice rib.	0	Seurig.	1894	" " "
26	St. John, N. B., Canada	550.0	97.0	5.6	17.7	30.0	25.0	60.0	4.5	D.	Hy.	Lat. Rib.	2	Schneider.	1916	Kunz.
27	Niagara, N. Y. Ry	550.0	120.0	4.6	21.8	30.0	80.0		5.8	D.	Hy.	Spand. Br.	2	Hinkley.	1916	*Engineering News*, 1916-B.
28	Niagara, Germany	548.0	78.0	7.0	14.2	30.0	30.0	90.4	5.5	D.	Hy.	Lat. Rib.	2	Ritter.	1897	*Engineering News*, 1896-B.
29	Segura, G. T. Ry	550.8	114.0	4.8	20.7	30.0	20.0		3.6	D.	Hy.	Erected as cantilever while using suspension.	2	E. L. Buck.	1897	*Engineering News*, 1896-B.
30	Washington, New York City§	543.0	84.0	6.0	15.5	35.0	15.0		2.8	D.	Hy.	Longest of designs submitted in competition.	2	Hutton.	1897	*Engineering News*, Dec. 27, 1900.
31	Cassini, France	541.3	126.6	3.9	24.5	32.0	28.9		6.0	D.	Hy.	Most striking design of 2-hinge deck lattice rib	2	Bauby.	1885	*Engineering News*, 1894-B.
32	Rosenanlage, Germany	540.0	90.0	6.0	16.7	30.0	14.0		3.6	H.T.	Hy.	Best example of light 3-hinge through lattice rib.	3	Worcester.	1905	*Engineering Record*, Nov., 1906.
33	Washington, New York City§	540.0	90.0	6.0	16.7	29.8	15.0		2.8	D.	Hy.	Design received second prize in competition.	2	Hildersbrand.	1897	*Engineering News*, Dec. 27, 1900.
34	Lessau, Germany	520.0	69.0	7.5	13.3	30.0	10.5	19.4	1.9	D.	Hy.	Riel Canal arch, of comparatively small rise.	2	Lauter.	1899	*Engineering*, Aug. 18, 1905.
35	Tarbin, Portugal	520.7	99.0	5.5	15.4	30.0	20.0	29.8	3.8	Hy. & Ry.	Spand. Arc.	Somewhat similar to Nos. 31 and 36.	2	Daly.	1910	*Engineering News*, June 15, 1905.
36	Ria Maria, Douro, Portugal	523.0	99.0	5.3	15.4	30.0	32.8		6.3	D.	Hy.	Very bold deck 2-hinge lattice rib of good rise.	2	Seurig.	1910	Mebrieau.
37	Cannon, South Africa	500.0	66.6	8.1	13.6	35.0	13.9	29.0	2.8	D.	Hy.	Design received second prize in competition.	2	Hochhoff.	1914	*Engineering News*, Dec. 26, 1918.
38	St. Louis, Mo	518.0	77.0	6.7	14.9	30.0	12.0		2.3	H.T.	Hy. & Ry.	Lat. Rib.	3	Eads.	1874	*Engineering News*, 1907.
39	Ehrenbreitstein, Germany	518.0	57.8	9.0	11.2	30.0	12.0		2.3	H.T.	Hy. & Ry.	Riel Canal. Spread of arch ribs at Canal.	3	Eggert and Grave.	1898	*Engineering News*, 1896-B.
40	Washington, New York City	510.0	91.7	5.6	18.0	30.0	18.0		3.5	D.	Hy.	Design built. With rise used, this should be deeper.	2	Hutton.	1899	*Engineering News*, June, 1867.
41	Coblenz, Germany	509.0	49.0	11.4	9.6	30.0	12.0		2.3	D.	Hy.	Ribs spaced at No. 39. Erected on falsework.	2	Gehr.	1874	*Engineering News*, 1907.
42	Kassel, Africa	500.0	90.0	5.6	18.0	30.0	15.0	105.0	3.0	D.	Hy.	Second to length in Africa. At Zambezi Falls.	2	Ende and Partners.	1905	*Railway Gazette*, 1908.
43	Kassel, Africa	500.0	67.0	7.5	13.4	30.0	25.0		5.0	H.T.	Hy. & Ry.	Truss Rib.	3	Ende Berger.	1909	*Engineering Record*, Oct., 1908.
44	Kassel, Paris	490.0	65.8	7.0	14.2	30.0	7.0		1.5	H.T.	Hy.	3-hinged braced arch. Type applicable to No. 61.	3	Lavoinne.	1905	*Engineering News*, Dec., 1906.
45	Washington, New York City	490.0	73.5	6.7	15.0	30.0	10.0		2.0	D.	Hy.	Design received first prize in competition.	2	Schneider.	1897	*Engineering News*, Dec. 27, 1900.
46	Kassel, Main	486.0	90.0	5.0	19.7	35.0	6.0	96.0	1.2	D.	Hy.	Spand. Br.	2	Seguil.	1888	*Engineering Record*, May 18, 1907.
47	Kassel, Costa Rica	480.0	72.0	6.7	15.0	30.0	12.0	24.0	2.5	D.	Hy.	Deck Arch. Sp. Br.	2	Cooper and Ass.	1902	*Engineering News*, C., 1905.
48	Kassel, Pittsburgh, Pa	480.0	75.0	6.0	17.0	30.0	15.0	14.0	3.8	D.	Hy.	Truss Rib.	3	Brunner.	1907	*Engineering News*, 1907-1.
49	Kassel, Colorado	465.0	93.0	5.0	20.0	30.0	20.0	40.0	4.3	D.	Hy.	Spand. Br.	2	Waddell.	1915	Waddell, Br. Engr.
50	Kassel, River, England	450.0	67.0	6.9	15.1	35.0	19.0	29.0	2.9	D.	Hy.	Spand. Br.	2	L. L. Buck	1890	*Engineering Record*, Aug., 1791.
51	Hayward, Ind.	400.0	40.0	10.0	10.0	35.0	10.7		2.7	D.	Hy.	Spand. Br.	2	Berne.	1896	*Engineering News*, June 23, 1900.
52	Nuremberg, Germany	378.0	57.5	6.6	15.2	30.0	11.8	28.2	3.1	H.T.	Hy.	Lat. Rib.	3	Nuremberg Works.	1902	*The Engineer*, Nov. 11, 1904.
53	Nuremberg, Germany	378.0	57.5	6.6	15.2	30.0	9.8	27.7	2.6	T.	Hy.	Truss Rib.	3	Freutzen.	1900	*Engineering Record*, Sept., 1.01.
54	Kassel, Kassel, Germany	378.0	70.0	5.4	18.5	30.0	11.0	19.4	2.9	D.	Hy.	Lat. Rib.	2	Arther and Fisher.	1907-8.	*Engineering News*, 1907-B.
55	Kassel, Kassel, Berne	377.0	70.6	5.4	18.7	29.0	17.0	40.3	4.5	D.	Hy.	Lat. Rib.	2	Rothlisberger.	1913	*Engineering News*, 1907-B.
56	Kassel, Kassel	367.4	69.3	5.3	18.6	30.0	9.0		2.4	D.	Hy.	Lat. Rib.	2	Lebert.	1919	*La Génie Civil*, Nov., 1918.
57	Kassel, Kassel	364.0	76.5	4.9	21.3	35.0	10.0	84.5	2.8	D.	Hy.	Spand. Br.	2	Shuck.	1926	*Engineering Record*, June, 1903.
58	Kassel, Hungary	360.0	72.0	5.0	20.0	30.0	9.0		2.5	D.	Hy.	Spand. Br.	2	Feierchany.	1904	Ass. Hra Ferra Hungarian.
59	Kassel, Minas, South America	350.0	72.0	7.0	14.4	35.0	9.0	84.0	2.6	D.	Hy. & Ry.	Cast. sp. Br.	2	Bender.	1894	Bender, p. 139.
60	Kassel, Bridge, Pittsburgh, Pa	350.0	70.0	5.0	20.0	30.0	4.0	49.3	1.1	D.	Hy.	Spand. Br.	2	Engineering.	1907	*Engineering Record*, June, 1898.
61	Kassel, Ill., Paris	330.6	97.6	3.4	17.0	5.9	20.0		1.4	D.	Hy.	Cast-Steel Gir.	3	Cast-steel vertices More rise possible. See type No. 44.	1892	*Engineering News*, April, 1898.
62	Kassel, Kassel, Germany	551.0	81.2	11.5	8.9	24.5	16.0		4.7	D.	Hy.	Very shallow 3-hinged deck arch. Good rib depth.	3	Altenloh.	1878	Mehrtens.
63	Kassel, Kassel, Germany	550.0	81.2	11.5	8.9	24.5	16.0		4.7	D.	Hy.	Spandrel are of grained rise. Less concrete rib.	3	Hilf.	1879	Turner.
64	Kassel, Kassel, Germany	463.0	53.3	12.8	8.1	24.5	15.4		4.7	D.	Hy.	3-hinged lattice rib, small rise. Illustrated in Mehrtens.	3	Hilf.	1879	Mehrtens, p. 248.
65	Kassel, Kassel, Germany	443.0	53.3	12.8	8.1	24.5	15.4		4.7	D.	Hy.	Type of crescent arch in Mehrtens.	3	Freutzen.	1901	*Engineering Record*, Vol. 59, p. 65.
66	Kassel, Kassel, Germany	341.0	75.8	10.4	9.8	24.5	6.9		1.9	D.	Hy.	Deck 3-hinged lattice rib. Small rise and rib depth.	3	Nuremberg Works.	1902	*Powerhouse*, 1902.
67	Kassel, Kassel, Germany	341.0	75.8	10.4	9.8	24.5	6.9		1.9	D.	Hy.	Crested cantilever over gorge. Very good proportions.	3	Modjeski.	1913	*Engineering News*, March 30, 1918.
68	Kassel, Kassel, New Zealand	340.0	42.0	8.1	12.4	30.0	7.0		2.0	D.	Hy.	3-hinged, plate-girder ribs of proper depth. Small rise.	3	Waddell.	1907	Waddell, Br. Engr.
69	Kassel, Kassel, Canada	327.0	42.0	7.8	12.8	30.0	7.0	35.2	2.0	D.	Hy.	3-hinged lattice rib of usual steel bridge details.	3	Fetterolf.	1908	*Engineering*, Aug., 1894.
70	Kassel, Kassel, Germany	335.9	62.9	10.9	9.7	30.0	4.9		1.5	D.	Hy.	Fl. Gir.	3	Competitive plan. Very shallow, 3-hinged, plate-girder arch.	1898	*Engineering*, July 19, 1901.
71	" " §	334.0	59.8	10.9	9.8	35.0	8.8	H.C.	1.9	D.	Hy.	Lat. Rib.	0	Competitive plan. Very shallow, 0-hinged, lattice-rib arch.	Lauter.	1898	" " "
72	" " §	334.0	59.8	10.9	9.8	35.0	8.8	8.8	1.9	D.	Hy.	Lat. Rib.	0	Competitive plan. Striking 3-hinged scheme with	Fagen.	1898	" " "
73	Kirshaus, Paris	328.0	39.0	10.1	9.6	35.0	19.0	9	D.	Hy.	Cast. Sp. Br.	Cantilever arch. Very small rise and rib depth.	2	Seual.	1896	*Engineering News*, Nov. 12, 1896.
74	Kassel, Germany	315.0	39.0	10.1	9.6	35.0	16.1		6.1	D.	Hy.	Half-through arch. Very small rise. Good rib depth.	2	Hawrich.	1895	Hawrich.
75	Kassel, Germany	307.0	31.0	9.9	10.1	35.0	4.0		1.8	D.	Hy.	Spand Br.	2	Krohn.	1899	*Engineering News*, 1896-1.
76	Kassel, Kassel, Germany	291.0	39.0	9.9	10.1	35.0	4.6		1.8	D.	Hy.	Lat. Rib.	2	Bridge.	1884	*Engineering Record*, Sept., 1898.
77	Kassel, Creek, Canada	290.8	39.0	9.8	10.0	35.0	16.0	65.0	5.5	D.	Hy.	Spand. Br.	2	Peterboe.	1896	*Engineering Record*, July, 1894.
78	Kassel, Kassel, Canada	290.0	52.0	9.8	11.1	35.0	17.8	69.2	6.1	D.	Hy.	Spand. Br.	2	Waddell.	1905	*The Engineer*, 1905.
79	Kassel, Kassel, Kassel	288.8	39.2	10.0	7.7	25.0	1.3	4.6	0.6	D.	Hy.	Fl. Gir.	2	Cudry.	1885	*Engineering News*, 1896-1.
80	Minneapolis, Minn	286.0	39.0	9.8	10.0	35.0	4.0	4.9	1.4	D.	Hy.	Fl. Gir.	2	Shedd.	1896	*Engineering Record*, Sept., 1898.
81	Spokane, Wash	256.0	32.9	7.8	12.8	35.0	8.0		3.4	D.	Hy.	Spand. Br.	2	3-hinged combination span. Small rise.	Fowler.	1904	Proc., Pac. N. W. Soc. Engrs.
82	Kinsville, Tenn	249.0	70.0	8.0	19.5	35.0	50.0		8.8	D.	Hy.	Arched Cantilever. Small rise. Large center depth.	2	Fowler.	1906	*Engineering News*, 1896-1.
83	Monte Video, Africa	240.0	40.0	6.0	16.7	35.0	8.0		3.4	D.	Hy.	Spand. rafter, spandrel-braced arch. Erected cantilever.	2	Fowler and Wood.	1911	Fowler and Wood.
84	Kansas, Kansas	240.0	41.0	6.8	17.1	32.5	6.0	8.0	2.8	D.	Hy.	3-hinged circular arch, lattice rib with tie	3	Fowler.	1914§	Fowler
85	Kansas, Kansas	240.0	41.0	6.8	17.1	32.5	6.0		2.8	D.	Hy.	3-hinged, lattice rib crescent arch. Erected in stages after service.	3	Bashr.	1905	*Engineering News*, Aug. 26, 1910.
86	Durban, Natal, South Africa	230.0	58.5	9.5	40.9	29.0	9.0		4.3	D.	Hy.	3-hinged rectangular arch erected as cantilever	3	Max am Ende.	1894	Mehrtens p. 664.
87	Kansas, Kansas	230.0	58.5	9.5	40.9	29.0	9.0	40.9	4.3	D.	Hy.	Spand. Br. Fl. Gir.	2	Tarrington.	1896	*Engineering Record*, Sept. 4, 1897.
88	Youngstown, Ohio	210.0	30.0	8.5	14.3	35.0	9.0		3.8	D.	Hy.	Fl. Gir.	2	Fowler.	1896	*Engineering Record*, 1898-1.
89	" "	210.0	30.0	8.5	14.3	35.0	9.0		4.1	D.	Hy.	Arch Cast.	2	Fowler.	1896§	Fowler.
90	Iron Mountain, Mich	207.0	54.0	4.0	26.1	35.0	11.0	41.0	5.3	D.	Hy.	Spand. Br.	2	Lowell.	1902	*Engineering News*, Nov., 1902.
91	Fairmount, P. rt, Philadelphia, Pa	200.0	40.0	5.0	20.0	35.0	4.8	44.8	2.4	D.	Hy. & Ry.	Spand. Br.	2	Four-span, 3-hinged, spandrel-braced. Good proportions.	Deane.	1908	*Engineering News*, Aug., 1898.
92	Fairmount, Ill.	200.0	40.0	5.0	20.0	35.0	4.8		1.2	D.	Hy.	Fl. Gir.	2	Only example of 3-hinged rib. Rib depth too small.	Lyglis.	1904	*Engineering News*, Dec., 1904.
93	Innsbruck, Austria	195.8	58.7	6.8	19.6	35.0	4.3	30.9	2.6	D.	Hy.	Spand. Br.	2	3-hinged, spandrel-braced arch. Fairly good proportions.	Hagen.	1899	*Engineering News*, 1900-1.
94	Fairmount, Pittsburgh, Pa	196.0	50.0	3.9	25.5	35.0	19.4	85.0	9.9	D.	Hy.	Fl. Gir.	2	3-hinged, plate-girder rib. Good rise and rib depth	Brunner.	1908	*Engineering Record*, 1900-1.
95	Hancock, Ky	193.8	45.9	4.3	23.8	30.0	6.6		3.4	D.	Hy.	Fl. Gir.	2	Excellent proportions. Estimated weight 125# over sp. br.	Hovey.	1911	American Bridge Co.
96	Hancock, Ky	193.8	45.9	4.3	23.8	30.0	6.6		3.4	D.	Hy.	Lat. Rib.	2	3-hinged, lattice rib. Deeper at quarter points of rib.	Bodin.	1909	*Engineering News*, Nov. 4, 1909.
97	Wendell, Ky	187.9	48.0	3.9	25.5	35.0	7.0	65.0	3.8	D.	Hy.	Spand Br.	2	Hovey.	1911	American Bridge Co.
98	Wendell, Ky	187.9	48.0	3.9	25.5	35.0	7.0		3.8	D.	Hy.	Tr. Rafter.	2	Hovey.	1911	*Engineering News*, April 28, 1910.
99	Passaic, New Jersey	180.0	51.9	3.5	28.8	35.0	9.0		5.0	D.	Hy.	3-hinged trues rafters. Erected in vertical position and lowered.	3	Sir C. deB.	1905	*Engineering News*, 1898-1.
100	Passaic, Niagara Gorge, France	172.2	37.7	4.6	23.0	35.0	9.5	9.0	1.5	D.	Hy.	Lat. Rib.	2	Baily.	1909	*Engineering News*, 1908-1.
101	Manhattan Drive, New York City	166.6	25.3	6.6	15.0	32.5	6.0		3.6	D.	Hy.	Well-proportioned 3-hinged, lattice-rib arch.	3	New York City.	1908	*Engineering News*, 1908.

*Three-hinged, fixed end. †Fixed end. ‡Square arch. §Designed, but not built. A. = Quarter point. L. = Span. R. = Rise. D. = Deck. T. = Through. H.-T. = Half through. Hy. = Highway. Ry. = Railway.

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(*d*).—Arch ribs with one center hinge, plate girders;
(*e*).—Arch with one center hinge, spandrel-braced;
(*f*).—Arch with two hinges, plate-girder ribs;
(*g*).—Arch with two hinges, lattice ribs;
(*h*).—Arch with two hinges, trussed ribs;
(*i*).—Arch with two hinges, spandrel-braced;
(*j*).—Arch with three hinges, plate-girder ribs;
(*k*).—Arch with three hinges, lattice ribs;
(*l*).—Arch with three hinges, trussed ribs;
(*m*).—Arch with three hinges, spandrel-braced;
(*n*).—Arch with three hinges, and fixed ends;
(*o*).—Arch with three hinges, no center depth;
(*p*).—Arch with five hinges, spandrel-braced;
(*q*).—Cantilever arches, three hinges;
(*r*).—Arched cantilevers, two hinges.

These eighteen types, of course, are only modifications of five basic types: no hinges, one hinge, two hinges, three hinges, and cantilever arches. The investigation as to the last word in economy for a given crossing, therefore, may prove to be an almost endless task, unless the engineer, from long experience, can single out two or three types which are quite sure to prove suitable. The chance of making an error therefore, would seem greater in the selection of the type, than the possibility of serious error in proportions.

It is the speaker's opinion that the economic depth can be varied 5% either way, without much exceeding the computer's most probable percentage of error in making an estimate from a stress sheet, and the variation of sectional area, which may easily total 5 per cent. This is undoubtedly borne out by a study of the author's diagrams.

The list of one hundred arches has been studied by tabulating the ratio of rise to span, as well as the depth of the ribs or arch, and an average result thus determined, which, in a great measure, is valuable as a composite of the investigation into these important features.

Plate-Girder Rib With No Hinges.—The average for the two arches of this type is only 7.2% of the span length for rise, or a ratio of $L \div R$ of 13.8; which, of course, is too shallow, and less than half the rise recommended by the speaker in Table 9. The average depth of rib at the center is only 0.9% of the span length, or about one-third of the minimum advised. In fact, the Arcole Arch, in Paris (No. 79), failed under modern traffic and had to be reinforced.

Lattice Rib With No Hinges.—The average of seventeen arches of this type is 22.6% of the span length for rise, or a ratio of $L \div R$ of 4.4; or, discarding three distinct violations of economy, 25.4%, or a ratio of 3.9, which latter value, however, is only about three-fourths of the author's average. The average percentage of L for

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depth of rib is 3.0, but, discarding six cases where the depth is manifestly too small, the average is 3.8, or only practically one-half the value given by the author.

Plate-Girder Rib With One Hinge.—There is only one example of this type, and it agrees closely with the author's values; the percentage of rise to span length is 21.8, or a ratio of $L \div R$ of 4.6; the percentage of L for depth of rib is only 1.5, which is undoubtedly much too low, even for a plate girder.

Spandrel-Braced Arch With One Hinge.—There is only one example of this type, the design for an 1 800-ft. voussoir arch span at Quebec, and the possible rise is too low for a deck arch. The percentage of span length for rise is only 9.1, or a ratio of $L \div R$ of practically 11.0; the percentage of L for center depth is only 1.2. The location was certainly more suitable for a half-through arch of the type shown in Fig. 24, (No. 5 in Table 8, Plate I), and described later for this same crossing.

Plate-Girder Arch With Two Hinges.—There are three examples of this type, and the average percentage of rise to span is 18.7, or a ratio of 5.3; but, discarding the very shallow arch, No. 70, the average is 23.3%, or a ratio of $L \div R$ of 4.3; the average percentage of L for depth of rib is 3.2, and the maximum is only 3.8.

Lattice-Rib Arch With Two Hinges.—The average for twenty-five arches is 17.1% of the span length for rise, or a ratio of $L \div R$ of 5.8; but, discarding seven distinct violations of economy, 20.0% or a ratio of 5.0; the first slightly low value is accounted for by the seven deck arches included, of only about one-tenth rise; the average percentage of L for depth of rib is only 3.8; but, if five of the largest values are excluded, the result would be only 3.2. The average of those omitted, however, is only 6.0%, and the maximum value is 6.3%, as compared with the 7.5% recommended by the author.

Spandrel-Braced Arch With Two Hinges.—The average of the twelve cases of this type is 17.1% of the span length for rise, or a ratio of $L \div R$ of 5.8; which value is perhaps slightly too low for economy, but is accounted for by a number of deck arches with insufficient distance below grade to allow for a greater rise; the average values found by discarding the very flat arches, are 18.5% and a ratio of 5.4. The average percentage of L for center depth is 3.4, with no normal value in excess of 5.5.

Plate-Girder Rib-Arch With Three Hinges.—There are only six examples of this type, and the average percentage of rise to span length is 18.6, or a ratio of $L \div R$ of 5.4; or, discarding the two very flat arches, 23.6%, or a ratio of 4.2. The average percentage of L for depth of rib is 2.8, or, discarding the two abnormal cases, 3.3, either value being high enough for plate-girder ribs.

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Lattice-Rib Arch With Three Hinges.—The average for fifteen arches is 20.2% of the span length for rise, or a ratio of $L \div R$ of 4.9; and, as the highest value is 27.3%, or a ratio of 3.7, this type would seem to agree more closely with the author's results. The average percentage of L for depth of rib is 3.6, for rational cases, or practically one-half the author's value; and the maximum value is only 4.8 per cent.

Spandrel-Braced Arch With Three Hinges.—The average for eighteen arches is 19.7% of the span length for rise, or a ratio of $L \div R$ of 5.1, which is only slightly less than the values recommended by the speaker in Table 9, but considerably less than those recommended by the author. The foregoing average value includes one high-deck arch in which the percentage is 35.5 and the ratio 2.8, which is the only really high-deck spandrel arch (No. 63) found recorded. The average percentage of L for center depth is 3.2, and 7.2 for the high-deck spandrel arch (No. 63).

Spandrel-Braced Cantilever Arch, Five Hinges.—The one design described in this discussion has a span of 1 670 ft. and a rise of 272 ft. The rise is 16.3% of the span, or a ratio of $L \div R$ of 6.1; the center depth is 2.1% of the span length.

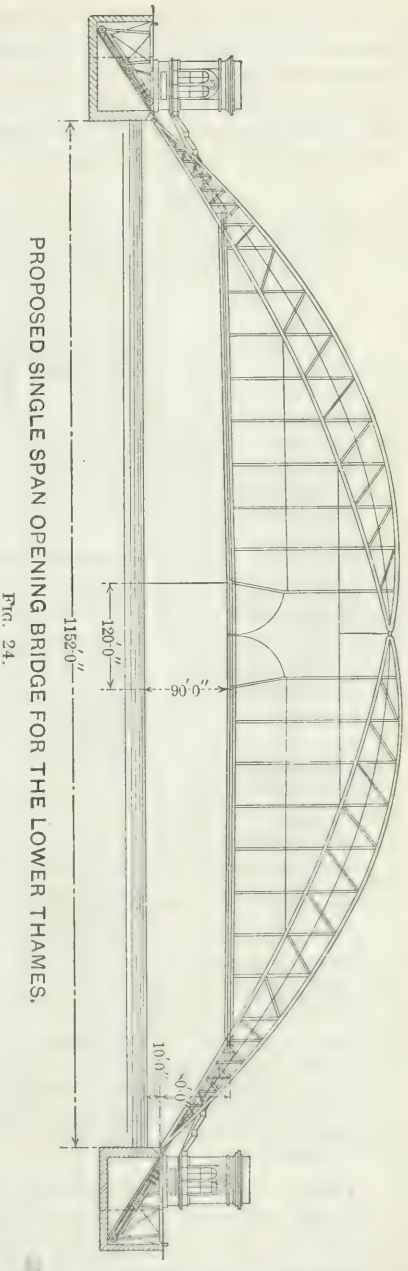

FIG. 24.

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The average for the one hundred cases is 18.9% of span for rise, or a ratio of $L \div R$ of 5.3; the average of the author for the same cases would be 26.6 per cent. The average for the one hundred cases (18.9%) indicates how much too low is the one-seventh or 14.2% general value given by Kunz; and that the author's values are too high, especially when applied to light highway and lighter railway arches.

TABLE 9.—Arch Bridge Rise and Rib.

Type.	Waddell's Ratios.			Existing Arches.			Fowler's Ratios.†		
	Light highway.*	Railway. Heavy highway.	Center and rib.	Light highway.	Railway. Heavy highway.	Center and rib.	Light highway.	Railway. Heavy highway.	Center and rib.
0-Hinge, deck.									
Plate girder	22.5	25.0	....	(2) 7.2	.....	0.9	17.5	20.0	2.5
Lattice rib	25.0	28.0	7.5	(1) 22.0	(7) 20.4	3.6	20.0	22.5	3.5
0-Hinge, high deck.									
Plate girder	30.0	33.0	....	.....	.....	....	20.0	25.0	3.0
Lattice rib	35.0	38.0	7.5	.....	(5) 33.0	3.4	25.0	35.0	3.5
0-Hinge, through.									
Lattice rib	30.0	33.0	7.5	(1) 23.5	(2) 24.0	1.0	22.5	25.0	3.5
1-Hinge, deck.									
Plate girder	20.0	22.5	....	(1) 21.8	.....	1.5	17.5	22.5	3.0
Spandrel braced	22.5	25.0	7.5	.....	(1) 9.1	1.2	20.0	25.0	4.0
2-Hinge, deck.									
Plate girder	17.5	20.0	....	.....	(2) 23.3	3.2	20.0	25.0	3.5
Lattice rib	20.0	22.5	7.5	(2) 18.6	(7) 20.6	3.8	22.5	28.0	4.0
Spandrel-braced	22.5	25.0	7.5	(3) 13.1	(10) 18.5	3.4	17.5	22.5	4.0
2-Hinge, high deck.									
Plate girder	22.5 25.0	25.0 28.0	....	.....	.....	....	....	....	....
	30.0	33.0	....	.....	.....	....	25.0	33.0	3.0
Lattice rib	35.0	38.0	7.5	.....	(2) 30.9	4.6	38.0	45.0	4.0
2-Hinge, through.									
Plate girder	20.0	22.5	....	.....	.....	....	17.5	20.0	3.5
Lattice rib	27.5	30.0	7.5	(2) 15.7	(9) 17.1	3.2	20.0	22.5	4.0
3-Hinge, deck.									
Plate girder	17.5	20.0	....	(2) 17.5	(4) 23.6	3.3	20.0	22.5	3.3
Lattice rib	20.0	22.5	7.5	.....	(2) 20.2	3.3	22.5	25.0	4.0
Spandrel-braced	22.5	25.0	7.5	(2) 17.1	(15) 22.0	3.2	20.0	22.5	5.0
3-Hinge, high deck.									
Plate girder	22.5 25.0	25.0 28.0	....	.....	(1) 30.4	3.3	25.0	35.0	3.0
	30.0	33.0	....	.....					
Lattice rib	35.0	38.0	7.5	.....	(2) 33.8	3.0	33.0	40.0	4.0
Spandrel-braced	....	....	....	(1) 35.5	.....	7.2	30.0	35.0	6.0
3-Hinge, through									
Plate girder	20.0	22.5	....	.....	.....	....	17.5	20.0	3.5
Lattice rib	27.5	30.0	7.5	(3) 15.8	(8) 18.7	3.7	20.0	22.5	4.3

* First column = Waddell less 2½%, his allowable variation.

† The ratios in the last three columns are those recommended, based on a study of existing arches, Waddell's ratios, and ratios in ordinary use, giving the minimum rise for deck arches and the maximum for high deck and through arches. Not based on three middle columns. Figures in parentheses refer to the number of normal cases averaged.

The average value of depth of rib for the one hundred arches is 3.2%, as compared with 7.5% of the span length recommended by the author; and indicates that the values given by the speaker in Table 9 are more logical, or a safe basis for investigations, as it will

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give a rib depth great enough for rational designing as well as economy. Mr. Hovey's investigations for the plate-girder arch ribs of No. 95 bear out the author's deductions for plate-girder ribs, as given in Table 9.

The Hell Gate Bridge proportions were fixed after a very careful investigation and analysis, and it is stated* by O. H. Ammann, M. Am. Soc. C. E., that:

"The weight of an arch of this type and length varies inappreciably for variations in rise between one-sixth and one-fourth of the span [or 8.3% range in rise], the absolute minimum being probably nearer the lower value [16.7%].

* * * * * * *

"The height of the trusses was decreased toward the center to 40 ft. 2⅝ in., or approximately one-twenty-fourth of the span length [4.1%], so as to reduce the temperature stresses, which are greatest at the center. The height of 140 ft. at the ends of the trusses was determined by the necessity for rigid portals between the end posts above the track floor. These assumptions for height resulted in the slightly reversed curve of the top chord, which produces a very pleasing sky line."

The speaker agrees with this last statement, and believes, also, that the proportions of this structure are scientific and perfectly logical.

The data by Kunz as to the proportions of steel arches are worth quoting, but are too general to be of great value. The ratio of rise to span is stated to range ordinarily from one-fourth to one-twelfth with possible extreme values of from one-half to one-fifteenth; and with an average for economy of one-seventh. (This the speaker has pointed out as being too flat, except possibly for very light highway bridges.) Kunz states that the center depth should range from one-sixtieth to one-fifteenth (or 1.7 to 6.6%), and he recommends for two-hinged, spandrel-braced arches, with parallel chords, from one-twenty-fifth to one-twentieth (or from 4.0 to 5.0%, which accords with the speaker's views); crescent ribs from one-eighteenth to one-fifteenth (or from 5.6 to 6.7%, which the speaker has pointed out to be very extreme values); with spandrel-braced arches, where the height is limited, the center depth may be as small as one-fortieth (2.5%), but the center panels should have solid webs. The foregoing may be reduced from 25 to 40% for highway bridges. Kunz also states that the depth of plate-girder ribs should be from one-sixtieth to one-fortieth (from 1.7 to 2.5%, which are certainly minimum values); and that the suspended span of cantilever arches should be from one-half to five-eighths of the side span.

Conclusions.—The conclusions reached by the speaker, from the foregoing average values, the author's values, and a study of the

* In his paper entitled "The Hell Gate Arch Bridge and Approaches of the New York Connecting Railroad Over the East River in New York City", *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 852.

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individual designs in detail, are that the following conditions should be observed:

Where the possible rise below grade would make the arch too flat for rigidity, as in numerous cases listed, notably the Alexander III Bridge, the Washington Bridge, the St. John Bridge in Canada, and the design for an 1 800-ft. arch at Quebec by Worthington, then a half-through type of arch would be preferable. Although the vibrations from the live load are not all due to the small rise, as has been shown in truss spans of extra depth built by the speaker, where the bouncing could only be attributed to the inertia of a heavy, paved floor, the greater rise recommended is certainly advisable.

Where there is a greater depth below grade, than is required for economical rise, the economy of the structure is but little changed by making the rise proportionately great, leaving no more, in any case, than room above the crown of the arch for a distributing truss, if it is to be used, and not carrying the roadway directly by long posts, which are unsightly when they extend above the top of the arch rib. The designers of the Viaur, Cleveland, Luiz I, Mungsten, Garabit, Pia Maria, Kornhaus, St. Croix, White Pass, Blauw-Krantz, Youngstown, Fern Hollow, Namti, and Surprise Creek Arches evidently found this method of design most artistic, and fairly economical, or they would not have used such proportions. The speaker has in mind no actual cases which would indicate a contrary view.

Wherever possible the rise should be greater than indicated by theory or ordinary practice, especially in the case of railway arches or those for heavy highway loads, and it is found that twenty-four of the arches in Table 8 (Plate I), have a rise of practically 25.0% or greater, and eight of these have a rise greater than 30.0 per cent. The study of individual cases indicates that the depth should seldom be less than 20.0%, and should preferably be 25.0% or greater, in the interests of stiffness and consequently longer life of the structures carrying the heavier loadings.

Where the best bridge is desired, that is, the one with the greatest ultimate economy, and not the cheapest in first cost, it will usually be found economical, and more artistic with a deck structure, to build it with a rise greater than 25.0%, and often up to 40.0% or greater, when there is room below grade; or even for a half-through arch when it would not prove to be top heavy.

Where the possible rise is materially less than 17.0% for heavy loading, some other type than a deck arch should be used, or the idea of an arch should be abandoned and a structure of an entirely different type designed.

The depth of rib, or center depth, as recommended by the author, would seem to be excessive; and, for arch ribs, the center or quarter depth should only be from 3.0 to 4.0% of the span length, except in

the case of solid ribs, where the depth might be as small as 2.5%, but never less than 2.0%, unless for some unavoidable reason, and greater than 2.5% for spans of small rise. Mr. Fowler.

The end depth of ribs without hinges, of course, must be greater than at the center or crown, as required by the stresses, but, for appearances, never less than one and one-half times the center depth, and usually not less than two and one-half times. The greatest ratio is four times, as in the great Mungsten, no-hinge, lattice-rib.

The center depth of spandrel-braced arches has varied all the way from 0.0 to 8.5%, but a depth of about 4.0% would seem to be economical and artistic, and a depth as great as 7.0 or 8.0% would often be decidedly displeasing.

The center depth of three-hinged ribs is perhaps more pleasing when not less than the greatest depth of the rib, but such a design is not the most economical, as will be shown later.

The center hinge in a spandrel-braced arch has been placed almost always in the lower chord, and this is a much better location, especially with respect to detailing the structure.

The list, Table 8 (Plate I), in which 74% of the structures are deck arches, and which is as complete as the data in the Engineering Societies Library afford, shows that for steel arches from about 200 ft. to 3 000 ft. span, there have been designed and built of the no-hinge type 19.0%; of the one-hinge type 2.0%; of the two-hinge type 40.0%; of the three-hinge type 39.0%; and, of all these, 71% have been of the arch-rib type and only 29% of the spandrel-braced type.

Therefore it might be concluded that designers have found the arch rib much more economical as well as more artistic than the spandrel-braced arch, and the types with no hinges and one hinge less economical than those with two and three hinges, although this quite certainly has been due, to some extent, to an unwarranted prejudice against wholly statically indeterminate structures. From the tabulation and other published data, the practical economy of the two-hinged and three-hinged types may be considered as equal.

The findings of M. A. Howe, M. Am. Soc. C. E., as to the no-hinge type being the most economical, and the two-hinge type next in economy, agrees with the speaker's experience; and, as to the relative economy of the two-hinged and three-hinged types, this is borne out by the author and by the investigations* of C. W. Hudson, M. Am. Soc. C. E., the arches being of the spandrel-braced type, of 20.0% rise and 3.0% center depth, having a span of 200 ft., and designed for a paved highway. The weight of the two-hinged arch was found to be 109 000 lb. and of the three-hinged arch 115 000 lb., or a saving of 5.5% in favor of the two-hinged structure. H. S. Jacoby, Assoc.

* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 20.

Mr. Fowler. Am. Soc. C. E., gives a comparison* of two-hinged and three-hinged designs for the 550-ft. Niagara Arch, and states that the three-hinged arch showed a saving of only 0.84% over the two-hinged type. The investigation of an arch, three-hinged for dead load and two-hinged for live load, showed 4.2% greater weight than the regular two-hinged type.

The speaker's investigations for this discussion have convinced him that the three-hinged, parabolic, lattice-rib arch, as shown in Fig. 24, is the most economical type of any given in Table 8 (Plate I), and it is certainly worth investigating for any long and important span. Table 8 gives the economical proportions for an arch of this type, for Nos. 3, 5, 6, 7, and 8. It indicates that the economical rise for heavy loads is from 19.3 to 23.5% of the span length, and the economical maximum depth of the rib as only from 4.3 to 4.8% of the span length, although the exact depth of rib is determined automatically for each case by the calculations. The development of this type was due to Mr. Max am Ende, the English engineer who designed the Blauw-Krantz Arch, South Africa (No. 86). The type was evidently first proposed in 1892 for the Thames River at London, just below the Tower Bridge.† The outline of the arch ribs is of most interest in this discussion, but the general scheme of this 1 152-ft. arch span is worthy of brief description. The roadway has a clearance of 90 ft. above the water and 80 ft. above the grade on shore, this difference of elevation being overcome by platforms operated on inclined tracks to elevate vehicle traffic to the level of the bridge roadway. All ordinary vessels could pass under the roadway, but, for vessels with tall masts or funnels, the double bascule opening of 120 ft. is provided. As the theory of this parabolic arch-rib structure has been fully developed‡ space will not be taken here, except to explain that the top flange of each half rib has the form of the curve of pressures for that half loaded, and the bottom flange has the form of the curve of pressures for that half unloaded; thus, both flanges are normally always in compression. All will remember the old bowstring truss, also the King and Massillon Arches, each of which closely approximated the outline of the moment curve, and was theoretically of the lightest weight for a simple span. Engineers still endeavor to make truss outlines, for economy, conform as closely as possible to the form of the curves of moments, and, therefore, it is not surprising that this parabolic rib arch should prove economical. It is in reality the Fidler truss for a suspension bridge reversed. Mr. am Ende has also proposed a suspension bridge of this type for the Straits of Messina, of 3 280-ft. span, which, as far as the

* Discussion of Mr. Hudson's paper in *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 31.

† This bridge is fully described in *The Engineer*, March 18th, 1892.

‡ *The Engineer*, April 14th and May 19th, 1893.

speaker knows, is the longest span of any kind that any one has had the temerity to propose seriously. Among the designs listed in Table 8 (Plate I), is one for the Firth of Forth of two 1 650-ft. spans, for which the weight estimated is 24 942 long tons (or 10 763 long tons in one arch span), as compared with the total of 51 000 long tons for the structure actually built. The weight of this arch design can only be approximate, but the formulas for the weights of the various portions of such spans are Mr. am Ende's Equations Nos. 10, 12, 13, 14, 23, 26, 34, and 35,* and would form a valuable basis for investigating this type. The other bridges proposed were for Quebec, of 1 600-ft. span, with a weight of 11 714 long tons (or 9 264 long tons for one arch span); for Sydney, Australia, of a 1 600-ft. span, with a weight of 11 786 long tons; and for the Bosphorus, in Turkey, a double-track bridge with a 1 240-ft. main span weighing 5 172 long tons, having a total estimated weight of only 6 072 long tons, as compared with 9 952 tons for a cantilever design, the live load being 5 600 lb. per lin. ft. of bridge. The speaker considers any type of structure which can make such a supposedly favorable showing to be worthy of investigation;

* *The Engineer*, April 14th, 1893.

Mr. Fowler.

PROPOSED DESIGNS FOR A BRIDGE OVER THE NORTH RIVER, NEW YORK.

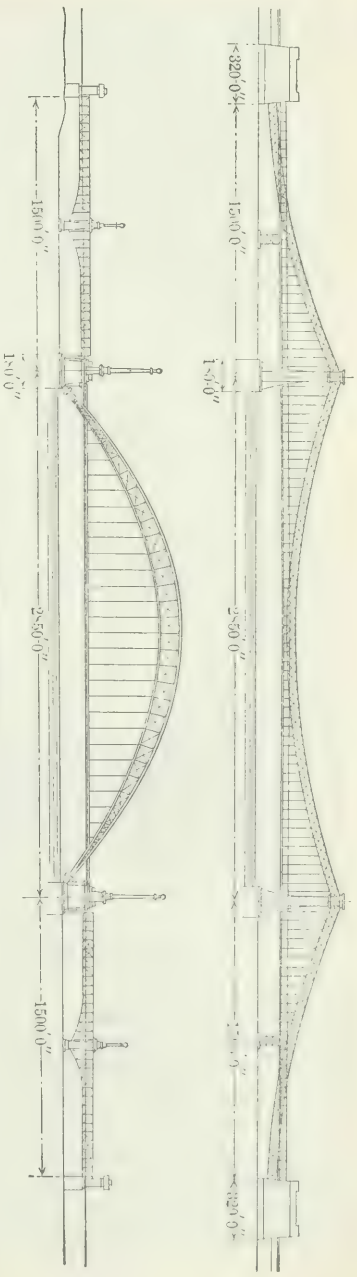
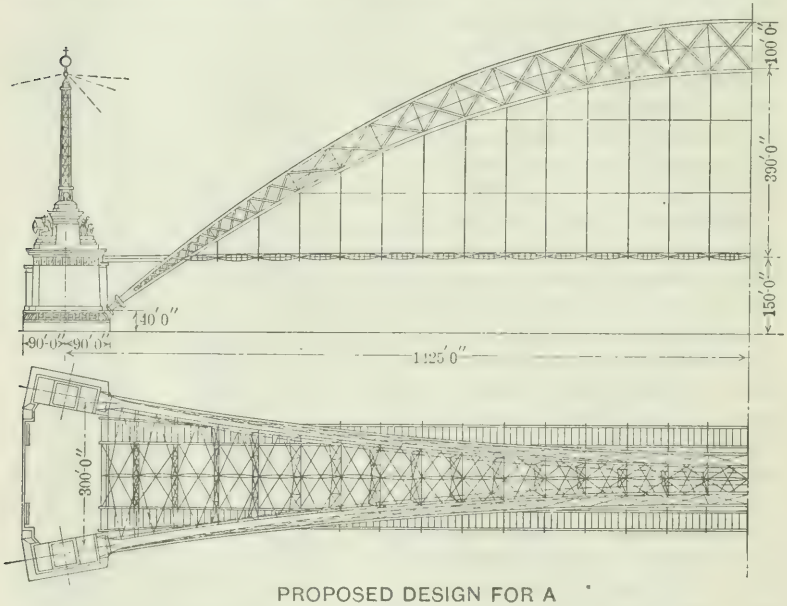


FIG. 25.

Mr. Fowler.

and this type, moreover, is much more graceful and pleasing than many other arch-rib types.

The same engineer proposed an arch span (No. 1) of 2 850 ft. for the Hudson River at New York, Figs. 25 and 26.* This, however, was a two-hinged sickle-shaped lattice rib of 440 ft. rise, which is only 15.5% of the span length, and indicates how top-heavy a through-arch of greater rise might become. The center depth of the rib, though only 3.5% of the span length, is 100 ft., and calls attention to the impossibility of using the 7.5% proposed by the author, especially for the longer spans. The statement of the author that 3 540 ft. is the maximum limit for an arch span, with present materials,



PROPOSED DESIGN FOR A BRIDGE OVER THE NORTH RIVER, NEW YORK.

FIG. 26.

would seem to be open to question, in view of such an unprecedented arch span having been quite carefully worked out by Mr. am Ende in 1889. The fact that proper foundations could not be secured, and that the comparisons with a suspension span are erroneous, does not detract from the value of his study and investigations.

The arch with five hinges, referred to in the classification, has not been included in Table 8 (Plate I), as it is a special type which, however, is deserving of mention. The design by Charles Bender for

* Fully described in *The Engineer*, May 17th, 1899.

two 1 670-ft. five-hinged arches for the Forth Bridge has been described at length by him.* He states that the weight was calculated twice independently with only 1% difference in the two estimates, so that the weight of 24 260 tons for the 5 320 ft. of bridge may be considered reasonably exact, to be compared with the 42 000 tons for the same length of that bridge, as actually built, and with the weight of 24 942 tons of the more logical three-hinged parabolic arch design (No. 3) of Mr. am Ende. The five-hinged design has two cantilever arms, each of 349 ft., with the arch chord pin-bearing at the springing or abutment, and supporting a three-hinged, spandrel-braced closing or suspended span of 1 002 ft. This is similar to the arrangement of the three-hinged fixed-end arches, Nos. 16 and 44, and is evidently worth investigating for long spans. The rise of the whole arch is 272.0 ft., or 16.3%, and 6.1 ratio of $L \div R$; the end depth is 278.0 ft., and the center depth 35.0 ft. or 2.1% of the span length. These figures denote a very well proportioned structure. Table 10 shows the proportions of all the designs for the Forth Bridge. Mr. Fowler.

TABLE 10.—Forth Bridge Designs.

Span, in feet.	Design.	Type.	Rise, percentage.	Ratio.	Rib, percentage.
1 710	As built	Cantilever,† two-hinged.....	7.8	12.2	3.5
1 670	Charles Bender ..	Spandrel-braced, five-hinged..	16.3	6.1	2.1
1 650	Max am Ende....	Lattice-ribbed, three-hinged..	20.0	5.0	4.3
1 640	Max am Ende....	Lattice-ribbed, no-hinged.....	24.0	4.2	1.2

† Simply included as a comparison of arched chord.

The arches, Nos. 81, 82, 83, and 88, in Table 8 (Plate I), are selected from those designed or built by the speaker, and are included in the list because all are quite striking, although they are not extremely long spans. The 255-ft. double-track railway arch at Spokane (No. 81, Fig. 27), as far as the speaker knows, is the longest spandrel-braced arch ever built with timber compression members. The design was made when C. T. Morris, M. Am. Soc. C. E., now of the Ohio State University, was Assistant Chief Engineer for the speaker, and it was planned to substitute steel compression members later for the two 10 by 24-in. arch timbers, and the other posts and struts in the structure, as the floor-beams, stringers, and bars were of steel. The erection of the arch presented no unusual difficulties, except that the erection superintendent failed to discover that the pin castings were not planed to the proper distance from the pin center, and the arch chord was consequently too long. This made the rise greater than intended (and the arch stronger), so permission was obtained

* "Principles of Economy in the Design of Metallic Bridges", New York, 1885.

Mr. Fowler.

to make all the diagonals turnbuckle members, and the extra camber was framed out in the ties.

The Knoxville arched cantilever, built by the speaker in 1896 (No. 68, Fig. 28), has been included as belonging to the cantilever arch type, and though possessing the desired artistic features, is possibly not as stiff under loads as even an ordinary cantilever. The spans were not made true arches after erection, on account of the great height of the piers.

The first design for the Youngstown Arch (No. 88, Fig. 29) was of the type of arch with cantilever end spans (No. 89) and as shown by the author in Fig. 14, but it was not found to be as economical as the two-hinged plate-girder arch as built. The stress sheets were made under the speaker's direction by E. M. Scofield, M. Am. Soc. C. E., then Principal Assistant Engineer of the Youngstown Bridge Company, and the calculations were made by Cain's graphic method. When checking up the stress sheets before construction, the late Professor A. H. Heller, then Assistant Chief Engineer, used Howe's analytical method, and found a practical agreement. The dead-load moment was 972 000 ft-lb., the live-load moment was 2 054 000 ft-lb., and the temperature moment 368 000 ft-lb. The structure had a paved deck, 60 ft. wide, the dead load being 6 200 and the live load 4 720 lb. per lin. ft. The load was carried to the arch ribs, 8 ft. deep, by posts supporting a lattice distributing truss, 10.5 ft. deep. The sub-trussing in the 165.5-ft. approach spans was made of the same depth, for harmony, and these spans were square-ended, 30 ft. deep, to give a proper backing to the arch, for appearance. This investigation indicates that all proportions of this structure were correctly determined. The speaker has found no other two-hinged plate-girder arches of more than 200 ft. span, except the Washington Bridge over the Harlem, and the Mayence design (No. 70).

The outline of the Switch-Back Arch on the White Pass and Yukon Railway, in Alaska (No. 83, Fig. 30) was made by the late H. S. Wood, M. Am. Soc. C. E., but, owing to a change in dimensions, was re-designed by the speaker before constructing it. The design is believed to be much more economical than a true or curved arch with long vertical posts, and, moreover, it was much cheaper to erect. It was designed for two 96-ton consolidation locomotives, with a train load of 2 700 lb. per lin. ft. The outline and graphics are shown on Fig. 31. The arches at Blauw-Krantz (No. 86) and at Namti Gorge, in China (No. 98), are of the same general type, the first named being what might be termed a no-hinged, rectangular arch, and the latter a three-hinged, trussed-rafter arch.

The subject is an absorbing one, and a volume could profitably be written describing the striking features of the arches on the list, but space prevents any further discussion, except to call attention

Fig. 27.—Spokane Railway Arch.

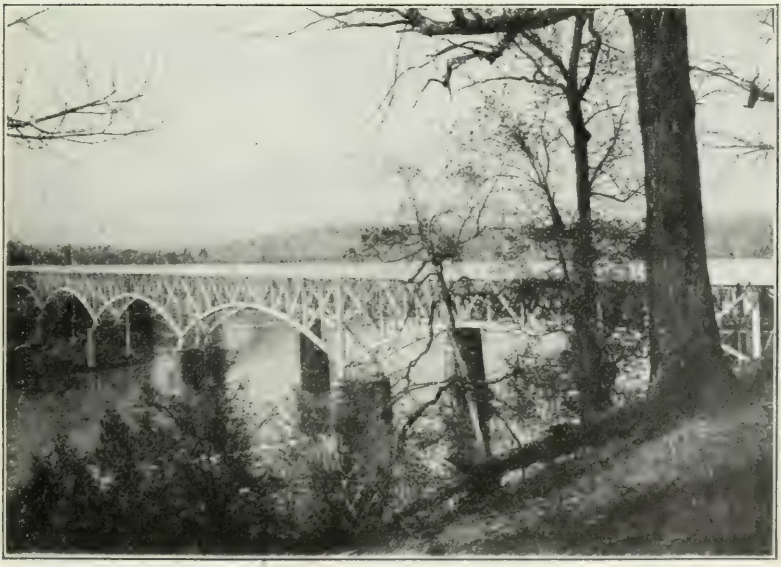


Fig. 28.—Knoxville Cantilever.

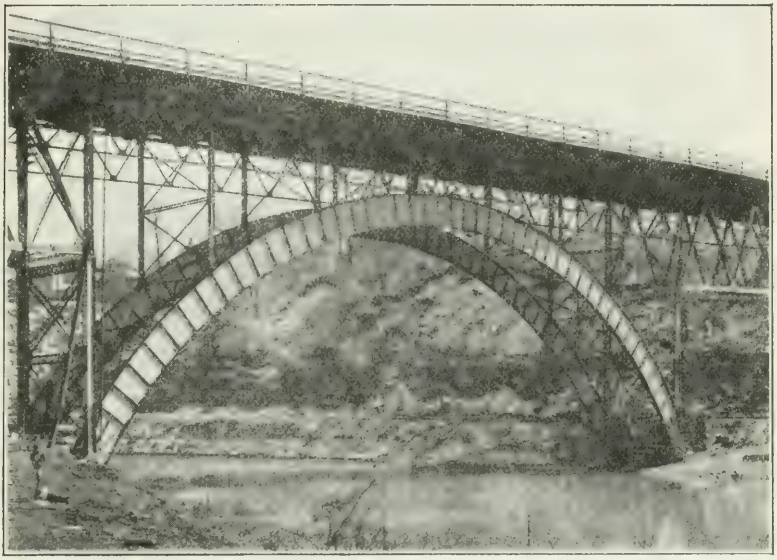


Fig. 29.—Mahoning River Arch.

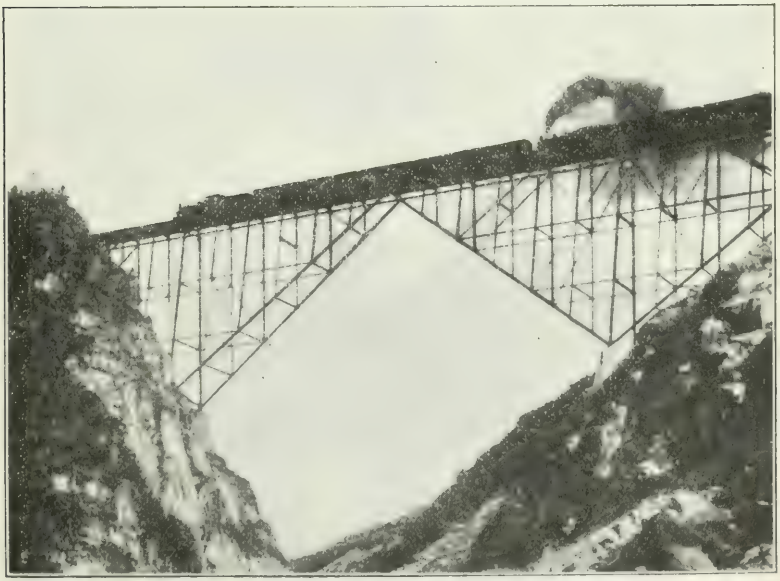
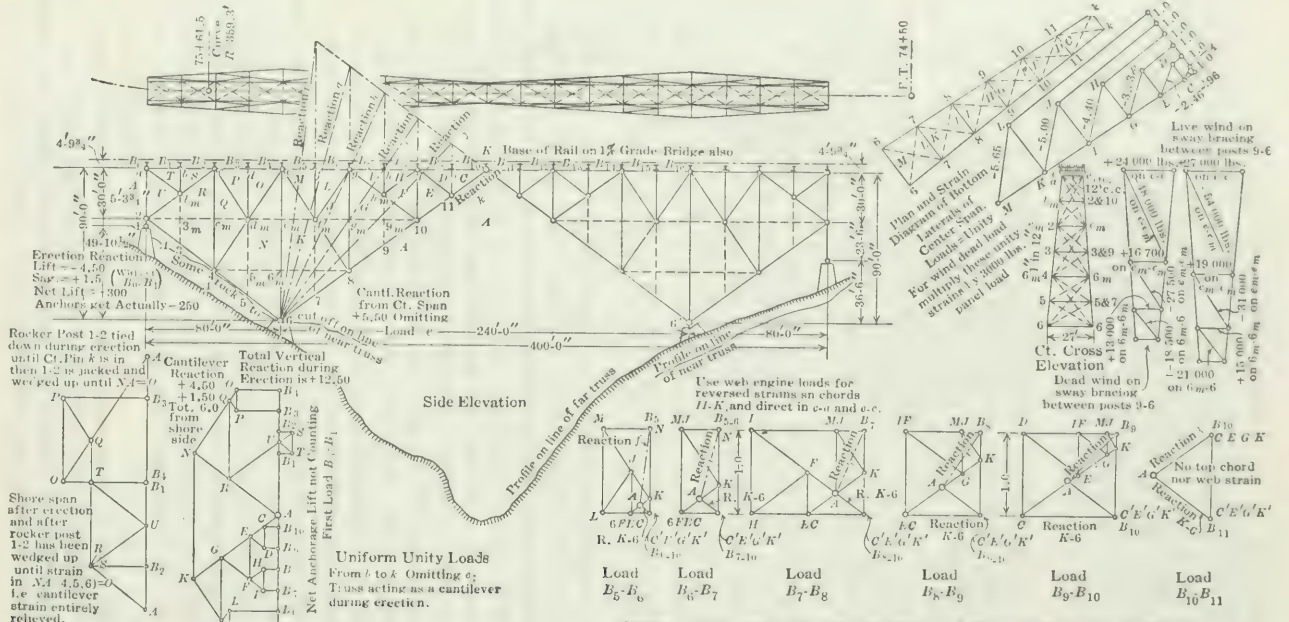


Fig. 30.—White Pass and Yukon Railway Arch.


Side Elevation
Base of Rail on 1% Grade Bridge also
Erection Reaction
Lift = −4.50
Sur. = +1.5 (Wt. ...)
Net Lift = +300
Anchors set Actually = 250
Rocker Post 1-2 tied down during erection until Ct. Pin K is in then 1-2 is packed and wedged up until X4 = 0
Cantilever Reaction +4.50
+1.50
Tot. 6.0 from shore side
Total Vertical Reaction during Erection is +12.50
Net Anchorage Lift not Counting
First Load B5-B6
Shore span after erection and after rocker post 1-2 has been wedged up until strain in X4 4,5,6 = 0 i.e. cantilever strain entirely relieved.
Uniform Unity Loads B1-2, B2-3, and B3-4 on shore span as a separate span.
Uniform Unity Loads
From b to k Omitting c; Truss acting as a cantilever during erection.
Cantl. Reaction from Ct. Span +5.50 Omitting
Load c
cut off on line of near truss
Profile on line of far truss
Profile on line of near truss
Use web engine loads for reversed strains in chords H-K, and direct in c-d and e-c.
Reaction
R. K-6
Load B5-B6
Load B6-B7
Load B7-B8
Load B8-B9
Load B9-B10
Load B10-B11
Separate "Unity Loads" Acting after Erection as a 3-Hinge Arch for Live and Dead Loads.
Plan and Strain Diagram of Bottom Laterals of Center Span. Loads = Unity
For wind dead load multiply these unity strains by 3000 lbs. panel load
Live wind on sway bracing between posts 9-6
+24 000 lbs. on each
+16 700 on each
+19 000 on each
+13 000 on each
+18 500 on each
+21 000 on each
Ct. Cross Elevation
Dead wind on sway bracing between posts 9-6
No top chord nor web strain

SWITCHBACK ARCH BRIDGE, WHITE PASS & YUKON RY., ALASKA.

FIG. 31.

Mr. Fowler. to the two three-hinged, fixed-end arches: the 626.9-ft. span at Vilaine, France (No. 16), and the 460-ft. span of the Austerlitz Bridge, in Paris (No. 44). They are the two most striking designs listed, and the latter proves how easily such a flat arch as the Alexander III Bridge could have been avoided.

The speaker's final words must call attention to the fact that the proportions fixed by him are average values for both short and long spans, and, though only small loss would result from their blind application to short spans—and ribs of too great depth would not be very unsightly for short spans—it is perceived, from a study of the longer arches, that it is often desirable to use a comparatively small rise, both as a measure of economy and as a prevention of top heaviness, and a comparatively small depth of rib, as one which is too deep would be uneconomical, unwieldy, and unsightly.

The conditions under which the load is applied to the arch rib may be a factor of economy, especially when a distributing truss is used and a considerable portion of the load is carried directly to the abutments. Then, too, the character of the steel will be a factor in fixing the economic proportions; and the entire problem of the design of large arches must be approached with an open mind, and a mind in full working order, to solve the particular case in hand.

Mr. Farr. W. B. Farr,* Esq. (by letter).—Due to the disadvantage of having only an abbreviated form of Dr. Waddell's paper, which abbreviation, it is stated, was necessitated by economy in publication owing to present-day conditions due to the war, and also to the fact that the sheets of graphic stress calculations, diagrams of stresses, and sections of metal, etc., and other particulars necessary for the full consideration of the paper, are not accessible, it is only possible for the writer to give the subject general consideration.

There can be no question that an arch bridge, whether the arch be of masonry or of metal, is more esthetic and pleasing to the eye than any form of girder or truss bridge with the usually straight bottom boom and rectilinear opening; though it cannot be denied that some bridges with trusses of large span by their very magnitude are impressive to the ordinary observer and quite beautiful to the trained eye of the engineer who alone can thoroughly understand and appreciate them.

A number of the large truss bridges erected in recent years in the United States are imposing and have the beauty of fitness to the eye of any one able to appreciate their design and construction. Any one who has stood on the track platform of the large bridge (the largest still of its type in existence) over the Firth of Forth, near Edinburgh, Scotland, and looked down the long vista of bracings,

* Chester, England.

intersecting in every direction, will be bound to admit that the sight is very imposing and beautiful, although the side elevation of the bridge, if imposing in its magnitude, cannot be said, even by one with a trained eye, to be a beautiful structure; it is, in fact, quite unnecessarily ugly. Mr. Farr.

Unfortunately, in this utilitarian age, the point as to whether or not a bridge will be esthetic, does not decide the matter, for the engineer is bound to consider his design from the point of view of cost, and whatever type of truss or arch, etc., is decided on, must be in most cases such that the bridge can be constructed at a minimum cost. Of course, there are exceptions, but the position stated usually decides the matter.

Dr. Waddell's paper is a most valuable contribution to the information available for the use of an engineer called on to decide not only the question of arch *versus* truss in his design for a new bridge, but also the question of the type of arch, whether the ribs should be of the two- or three-hinged type and with or without spandrel bracing or without hinges and with fixed ends. Lists of weights, etc., are given for all these types and will be found to be very useful, the information available in the literature of the Profession on the subject being very meager. The economic ratio of rise to span, economic depth of rib, and location of the crown hinge in three-hinged arches are all fully discussed, and valuable particulars are given.

The tables, diagrams, etc., even as given in the abbreviated form, are interesting as closely agreeing with the results of similar problems worked out independently, but not in such detail, by other engineers. It is not quite clear why Dr. Waddell, in Tables 4 and 6, gives the total weight of metal in pounds per linear foot "per side" for ribs and lateral system, while, in Table 5, he gives it in pounds per linear foot "of structure"—which appears to be a more desirable manner of stating it than "per side"—and perhaps he will be good enough to give some explanation of the matter.

The three-hinged arch with spandrel bracing will probably be chosen in most cases, owing to the stresses and reactions being determinate and ascertainable more readily and accurately than in the case of the two-hinged type or the type with fixed ends, because in these cases before an engineer can attempt to determine the stresses and reactions, he must make certain assumptions as to the elasticity of the superstructure and of the abutments, which may be sufficiently correct in his own estimation, but not so in the judgment of others, and which hypothesis cannot be absolutely demonstrated to be correct. In the case of the three-hinged arch, however, the locus of the curve of the center of pressure being definitely known at the three-hinged points, it is, therefore, definitely known at all other points

Mr. Farr. in the rib. Stresses in the superstructure due to variations of temperature, are eliminated in the three-hinged type, and these conditions will quite overbalance the defects due to the slight want of stiffness and greater vibrations under a moving load. It is quite inconceivable that a railway train would be allowed to cross such a structure at a high rate of speed, while in a highway bridge the vibration would be quite negligible.

The economy in the weight of metal in the arch over the simple truss appears, from the information in the paper, to commence at spans of more than 1 000 ft., but no comparison with the continuous truss of varying depth (of which the Forth Bridge is an example) is given. In bridges of several spans of more than 700 ft., a continuous truss of this type will be found to be much lighter in weight than any other truss or arch.

As it is only in the larger spans of more than 1 000 ft. that any economy in weight is shown over even a simple truss, and as the span at which the arch will come out lighter in weight than the continuous truss will be very much more than 1 000 ft., and as opportunities to construct such bridges are, especially in the Old World, of rare occurrence and not likely to enter into the experience of any but a few engineers and those in the highest ranks of the Profession, consequently, the average engineer is not likely to have an opportunity of constructing such a structure and can only as, in the writer's case, take an academic interest in the problems so ably dealt with by Dr. Waddell, to whom the thanks of the Profession both in the United States and England are due. The labor involved in the preparation of the paper and its attendant diagrams and tables must have been enormous.

In deciding on the type of bridge, in addition to the actual weights of metal in the superstructure, there is the question of abutments, nature of foundations, transportation and erection, and cost of maintenance and painting (which are too often overlooked in making an estimate of cost, as this work of maintenance, etc., frequently falls on shoulders other than those of the man designing the structure). The weight of the bearings at the ends of the three-hinged arch will vary according to the weight on them and, also requires to be taken into account, as by reason of the difficulties of, and the care required in, fixing, they may cost a considerable amount. Arches in metal are not suitable for a series of spans in a long and lofty viaduct, as it would not be possible to construct them satisfactorily on high braced or masonry piers, on account of the unbalanced thrust due to advancing moving loads over the spans. Construction of this type in such positions would probably never be seriously considered, but such viaducts are eminently suitable for crossing river gorges of comparatively narrow widths, with rocky sides forming suitable abutments

to take the horizontal thrust of the arches—such as those already erected at Niagara, or Victoria Falls on the Zambesi, over the River Douro, at Oporto, Spain, and the Garabit Viaduct in France—where it is easy to erect the arch ribs by tying them back to temporary abutments during erection, and where it would be difficult and expensive to erect a simple truss bridge. Mr. Farr.

HORATIO P. VAN CLEVE,* ESQ.†—Some one has said, "Genius is the capacity for doing work", and the speaker has had the truth of this assertion very forcibly impressed on him in reading this paper. In his preface to "Bridge Engineering" the author says that it was due to the World War, and the consequent stagnation in bridgework, that the writing of that treatise was made possible. It is probably no less true that the work on this paper could hardly have been done during the stress of the author's usual professional activity. Mr. Van Cleve

Some idea of the extent of the calculations incident to the preparation of the paper may be gathered from the statement—appearing as a foot-note under Problem No. 1—that seven designs were made for each of two loadings and presumably three types of rib, forty-two in all, in order to furnish data for that problem. Some short-cuts were undoubtedly made use of, but a casual consideration of the number of cases treated leaves little to be imagined regarding the extent of the labor involved.

The author's method of attacking the problems, namely, that of calculating quantities from actual designs, although more tedious, is far more practical than any wholly theoretical treatment, owing to the large number of variables involved. The latter method of investigation might perhaps have been pursued, but it would probably be more valuable as an exercise for the mathematical gymnast than as a working basis for the practical designer.

The form of the paper, although of secondary importance, deserves perhaps some attention at the hands of the critic. The speaker wishes to suggest that the matter presented would be more easily followed, and would be more convenient for reference purposes, if each natural division were prefaced by a re-statement of the problem to be solved, and, similarly, if appropriate notations were used for marking the subdivisions of the problems.

The speaker is interested in the author's proposal, in Problem No. 6, to make the arch rib three-hinged for dead load, and two-hinged for live load. It appears that a slight saving of metal would result from this expedient. To place a field splice across the crown-hinge would undoubtedly be possible, but, in the speaker's opinion, would rarely be justifiable, owing to the quantity of splice metal required, and the expense of the field drilling.

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* New York City.

† Now M. Am. Soc. C. E.

Mr. Van Cleve.

The chief value of the paper, as the speaker sees it, lies in the fact that it forms a basis of operations in steel arch calculations, guiding the designer's judgment, and thereby eliminating much of the cut-and-try work often resorted to, only to be discarded. In filling this much felt want, it will, no doubt, fully accomplish the author's purpose.

Mr. Jacoby.

HENRY S. JACOBY,* ASSOC. AM. SOC. C. E. (by letter).—The author deserves the hearty appreciation of the Profession for offering for publication the results of his elaborate investigations on certain economic relations of steel arch bridges. Only those who have undertaken similar work can comprehend what a vast amount of labor was involved.

Since the construction of the two arch bridges over the Niagara River, completed in 1897 and 1898, respectively, steel arches have been a subject for original investigation by many graduate students and some undergraduates in the Bridge Department of Cornell University. Brief statements of some of the most important results have been published.†

In order that the results might have more than an academic interest, actual designs have been used as the basis for comparative designs made under various conditions, in order to determine the relative weights and deflections of steel arches of different types.

A comparative study of different steel arch bridges actually built may not permit definite conclusions to be drawn in regard to economic design, because the loading, unit stresses, and other items of the specifications differ materially, and there are variations likewise in the form and proportions of the arches, and in the type and weight of floor systems. It is instructive, however, to consider certain facts related to past practice in the construction of steel arch bridges.

Eleven years ago Mr. C. H. Mallison compiled a table of about eighty American bridges containing metallic arches, with spans varying from 33 to 840 ft. About 85% were highway bridges. A study of this list shows that the number of arch bridges with three hinges is very nearly equal to the sum of those containing two or no hinges. In those with three hinges there are from three to four times as many having open webs as solid webs, and, for those without hinges, this relation is reversed in nearly the same proportion. Of the arches with two hinges, the number having open webs is more than

* Ithaca, N. Y.

† "Text-Book on Roofs and Bridges", by Merriman and Jacoby, 3d ed., Part IV, Arts. 85, 92-95.

Discussion on paper by C. W. Hudson, M. Am. Soc. C. E., entitled "Comparison of Weights of a Three-Hinged and a Two-Hinged Spandrel-Braced, Parabolic Arch", *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 34.

An article on the design of a two-hinged arch rib, *Transactions*, Assoc. Civ. Engrs. of Cornell Univ., Vol. VII (1898-99), p. 89.

An article on an investigation of comparative deflections of steel arch ribs with three, two, and no hinges, by Mr. Phoo Hwa Chen, *Cornell Civil Engineer*, Vol. XXVI (Feb., Mar., 1918), pp. 184, 229.

two-thirds as large as those with solid webs. Of the arches having open webs, more than three-fourths of the three-hinged arches are spandrel-braced, but only one-half of the two-hinged arches are of that construction. It would be interesting to know whether these general relations have been changed materially by the bridges built during the past decade. Mr. Jacoby.

Of the eighty bridges referred to, 25% have a span of less than 100 ft., 35% from 100 to 200 ft., about 20% from 200 to 300 ft., and about 20% have a span of more than 300 ft. The large number of short spans is due to the fact that steel arches have been used in parks and in cities where esthetic rather than economic reasons governed the choice of the type. In general, as the spans decrease in length, the use of solid webs increases.

Mr. Mallison found the proportions to be as given in Table 11, all values being expressed as percentages.

TABLE 11.

No. of hinges.	Character of web.	Ratio of Rise to Span.			Ratio of Crown Depth to Span.		
		Max.	Min.	Average.	Max.	Min.	Average.
3	Open.....	37.5	9.5	18.3	6.0	1.3	3.1
3	Solid.....	30.3	8.2	15.3	5.1	1.3	3.2
2	Open.....	27.8	7.1	17.6	6.7	2.2	3.7
2	Solid.....	28.6	8.0	14.1	4.2	1.9	2.7
0	.....	21.7	7.0	12.2	6.1	1.4	2.8

As the maximum values for the rise range from nearly four to about three times the corresponding minimum values, the effect of local conditions at the sites is indicated clearly. It will be noticed that the average values are highest for arches with three hinges and lowest for those without hinges.

The great range in crown depth from maximum to minimum seems hardly to be justified, either by economic considerations or local conditions, and serves to show the need for such investigations as have been made by Dr. Waddell.

On comparing the values in Table 11 with those determined by the author on the basis of economy only, it appears as if the latter values may serve a useful purpose as maximum limits when making a design for any particular locality.

Slightly more three-hinged arches have parabolic chords or axes than circular ones, but, for two-hinged arches, this relation is reversed. Nearly all the arches without hinges have circular axes or chords. Apart from the curves named, the straight line, hyperbola, and ellipse are used in only a few exceptional cases. The modern tendency is to increase the use of the parabola.

Mr. Jacoby.

Eighteen years ago, in discussing a paper on spandrel-braced arches, by C. W. Hudson,* M. Am. Soc. C. E., the writer recommended that two-hinged arches be designed and erected so that the dead-load stresses would be distributed as in three-hinged arches, and the live-load, impact, and temperature stresses as in two-hinged arches. This is accomplished by closing the arch with a temporary hinge and afterward transforming it into a two-hinged arch, at the proper temperature, when the structure is otherwise complete. In a spandrel-braced arch, this can be done by inserting the final member, without stress, at the mean temperature, and by riveting the connecting plates where the temporary hinge is located. By thus eliminating stresses due to inaccuracies in shop work, and due to erection, a higher degree of economy may be secured. It is gratifying to observe that this method has been used in practice in a number of notable examples. In some cases, however, the transformation to the two-hinged type was made before the floor system was put in place.

A study was made by Mr. R. W. Parkhurst to ascertain the influence of the position of the crown-hinge on the deflections of a three-hinged, spandrel-braced arch. For this purpose he used special designs based on the proportions and loads of the Niagara Arch Bridge, having a span of 550 ft. There are sixteen panels in the span. For convenience, let *B*, *I*, and *T* represent the three cases when the crown hinge is in the bottom chord, intermediate between the chords, and in the top chord, respectively. At Panel point 4, the quarter point of the span, the deflections under full live load are found to be 22 and 72% greater for Cases *I* and *T* than for Case *B*. At Panel point 6, the deflection for Case *I* is very slightly less and for Case *T* slightly greater than for Case *B*. At Panel point 8, the center of the span, the deflections for Cases *I* and *T* are, respectively, 40 and 55% less than that for Case *B*. The best form of deflection curve is given when the crown-hinge is between the top and bottom chords.

For a concentrated load placed at the center of the span, the deflections for Case *I* are intermediate between those of Cases *T* and *B* at all panel points, the greatest deflections being those of Case *B*. At Panel point 8 the deflections for Cases *I* and *T* are, respectively, 27 and 42% less than that for Case *B*.

When a concentrated load is placed at Panel point 4, the quarter point of the span, the differences between the deflections throughout the span are comparatively small. At the load, the deflection for Case *I* is almost midway between those of Cases *B* and *T*, that for Case *T* being the largest. At Panel point 8, the center of the span, the deflection is downward for Case *B* and upward for Cases *I* and *T*. At Panel point 12, the third quarter point of the span, all the deflections are upward, that of Case *I* being the largest, and that of Case *T* being the least.

* *Transactions*, Am. Soc. C. E., Vol. XLIII (1900), p. 34.

H. S. Sales,* Esq. (by letter).—It would be difficult to over-estimate the value of the information given in this paper. To the engineer who is first confronted with the problem of designing a steel arch, innumerable difficulties present themselves; and these have to be studied and solved *ab initio*. In a country like India, where steel arch bridges are seldom, if ever, used, it is unlikely that the engineer will be able to obtain much assistance in designing one, either by the advice of his colleagues, or by reference to recorded experience. One of the first questions he will ask is whether or not his preliminary layout is economical. It would be impossible to arrive at a decision on this vital point unless he had at his disposal sufficient time to work out a considerable number of skeleton designs, which is extremely unlikely. The labor involved can be better imagined than described in detail, as it is well known that arch calculations, compared with simple truss calculations, are extremely intricate; and it requires considerable practice to gain any facility in the work. With the aid of Dr. Waddell's notes and charts, the engineer's labors, however, will be lightened considerably, and, in fact, to such an extent that he will be able to begin the detailed design of an arch bridge systematically, provided he has had experience in steel structural work, and has a few type designs representing modern practice to which he can refer. Dr. Waddell remarks: Mr. Sales.

"Occasionally, an American bridge engineer builds a steel-arch structure; but usually simple-truss spans are adopted because their approximate weights and their economics are on record."

If this applies to America, it may be supposed that the same conditions prevail in India. The reason is obviously due to the circumstances under which engineers in India are necessarily placed. As mentioned previously, the beginning of steel arch design, leaving out the question of experience, necessitates an amount of study and time which the engineer cannot usually devote to the subject. Most engineers realize this, and would not willingly accept the task of designing an arch bridge unless sufficient time was available for carrying out the preliminary work thoroughly and systematically. Any information, therefore, which will result in a reduction of labor connected with the complicated preliminary calculations will be most gratefully received, and the thanks of the Profession in India are due to the author for his valuable paper, and the assistance which is to be derived by studying it.

The economy of steel arch bridges is established beyond question. The price of steel has gone up by leaps and bounds, and it may be safely assumed that finished structural steelwork cannot, in the near future, be obtained for much under Rs. 300 per ton. Within the next few years, therefore, it is reasonable to suppose that proposals may

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Mr. Sales.

be put forward for steel arch bridges, when conditions justify their use. It is much to be regretted that space could not be found to publish with the paper the graphic calculations of stresses, diagrams of stresses and sections, and tabulated estimates of metal, etc. The information would be valuable and instructive as examples of preliminary calculations and designs for reference. As an instance in support of this statement, take the extreme case of a hingeless steel arch: The estimation of the stresses in structures of this kind is anything but straightforward work, except, perhaps, by experts. Quantities have to be assumed tentatively, as, for example, the moment of inertia of the rib at various points. Hence correct judgment plays a very important part in the preliminary investigations; and, without it, the uninitiated is likely to waste a lot of time before he begins the actual design of the structure. Also, it is beyond question that the expert discovers for himself methods by which close approximations can be made, thus reducing to a minimum the trial-and-error part of the work. It is probable that the unpublished matter contains many shortcuts of the kind just mentioned, and that the paper would be more valuable if this part of the investigation work had been included.

The writer thinks that few experts in the design of steel arch bridges are to be found in India. The author himself draws attention to the need for special knowledge of this branch of bridge engineering, and even goes so far as to say that comparatively few structures of this kind have been built in America. There are certainly very few, if any, steel arch railway bridges in India. It would seem, therefore, that the author has undertaken the task of educating engineers in this particular branch of the profession; and the writer regards this paper as a successful effort in this direction. His only regret is that it could easily have been more complete. Nevertheless, it is hoped that, judging by past experience, the author will find time to write a treatise on arch bridges, and include in it all the diagrams and preliminary work for which space could not be found in his paper. These remarks lead to the consideration of an aspect of steel arch bridge design which, in the early days, was considerably neglected, but which, nevertheless, now seems to be receiving the attention it deserves. The writer refers to the peculiar knowledge possessed by the expert, namely, that which is concerned with the possible lines on which steel arch bridges can develop by alteration of type or shape of structure. The important question which he so frequently asks himself is: Is bridge design wholly science; or should it be largely an art? An enthusiast in any constructive science, the engineer included, will use all the means at his disposal to effect his object, whether these means consist in mechanical skill or the application of mathematical knowledge. In steel bridgework both are essential, and perhaps in some work, notably the arch bridge, mathematics plays the

most important part. Unfortunately, mathematics is regarded by many as the beginning and end of all bridge design; so that many engineers have, at last, been deterred from advancing on certain lines of progress because they felt that they could not undertake the enormous amount of computation work necessary before the actual scheming could be put in hand. To these the actual design had necessarily to occupy a secondary place which could not be approached without the mathematical knowledge required to complete the preliminary work. If, therefore, the mathematics of steel arch bridges can be presented in such a way that it constitutes a well-made tool from which further operations can begin, then the engineer is at once raised to a new level, and a strong inducement is provided for him to undertake work which, otherwise, he might not be able, or inclined, to begin. The greatest thanks of the Profession, therefore, are due to those who, like Dr. Waddell, have presented their mathematical work in tabular or graphic form suitable for general use. By doing so the mathematical part of the work has been reduced from a problem to a process, with the result that the engineer can devote more of his energies to that part of his work which may be described as "constructive art." Mr. Sales.

Problem No. 1.—On page 4 the author mentions "the lateral system in a simple truss bridge." There would seem to be some uncertainty as to what is exactly meant by this expression. Truss bridges are of two kinds, deck and through; and the latter is provided with a floor system in addition to the lateral system proper. Is it to be understood that the weight of the lateral system plus rail-girders plus cross-girders will offset that in the bracing of the floor system plus that of the rib flanges of the corresponding arch structure, plus that of the other diagonals between the arch ribs? Probably not; but the writer thinks that the fact might have been stated; or, what is still better, that the style of the structures in which the relative weights of parts are considered might be illustrated by clear diagrams. To bring Dr. Waddell's results into line with Indian conditions, much work will be necessary in connection with tabulating the weights of various types of spans designed for all gauges, and under different loadings.

The author refers to a very interesting point concerning a method for hinging the crown of two-hinged and three-hinged arches. His method is to hinge the crown at mid-depth for the dead-load stresses, and afterward to make both the upper and lower chords hingeless for the live-load stresses.

Problem No. 2.—The formula, $A = \frac{T + \frac{2M}{d}}{p_c}$, for the area of the

Mr. Sales.

chords, would seem to imply that the section will not be symmetrical about the neutral axis; and this, in fact, is what is found in the bottom raker struts and other principal members of certain cantilever spans where the top flanges are of greater area than the bottom ones. It would be interesting to know whether the contour of the arch rib can be placed so as to give symmetrical rib sections, and, if not, what rules are adopted to determine provisionally the position of the neutral axis.

The author frequently refers to the esthetic aspect of bridge design. His remarks lead one to regard esthetics as probably embodying economical principles hitherto unsuspected, except, perhaps, by experts. Whether mathematical considerations will eventually lead to true esthetics, or esthetics will indicate the true mathematical principles to be followed, are matters for speculation. It would be interesting to know what the author has to say on this subject. It would seem that the parabolic curve, so frequently adopted for the contours of arch ribs, is not a line of beauty, and that, under certain restricted conditions, arches might be reasonably made of circular form. The writer does not know whether this is generally possible, but thinks that the subject is worthy of investigation, as a slight increase in weight of metal might be compensated for by the better appearance of the bridge. Though there must be diversity of opinion on matters which are so often decided by individual taste and preference, yet the esthetics of bridges concerns every one, and, in such cases, guiding principles must be sought for, rather than an appeal made to what, after all, may be only traditional belief or sentiment.

Problem No. 3.—The placing of a hinge in a spandrel-braced arch is a problem which has been solved by the author in a very interesting manner. He points out that the conditions in light bridges are such that top chords are provided with sections which are in excess of theoretical requirements. By placing the hinge in the top chord, the superfluous metal is utilized, and the total weight in arch metal is considerably reduced. These are principles of construction which, to some extent, apply to simple-truss spans. It would be interesting to know how Dr. Waddell would design the ordinary through type of truss span so that the top lateral bracing could be arranged to assist the top chords to the utmost extent by taking a part of the compression to which these members are subjected. There seem to be strong grounds for believing that it actually does perform this office.

Problem No. 4.—The writer notes the author's conclusion that the spandrel-braced arch is lighter than the braced-rib structure for a 200-ft. span, but a little heavier for the 500-ft. span. The author, however, does not inform the reader whether the comparisons are made with hingeless, two-hinged, or three-hinged arches. Perhaps the relative weights are the same for all three types; but the point might have

been brought out in greater detail, and the part of the paper devoted to Problem No. 4 expanded considerably. Mr. Sales.

Problem No. 5.—The solution of Problem No. 5 is valuable. One of the first questions the bridge engineer will ask himself is: Is it justifiable to undertake an enormous amount of computation work to obtain stresses in a hingeless arch when perhaps, everything considered, the two-hinged or the three-hinged arch might, in the long run, be the most economical in metal and, from a practical point of view, be the best? The reader, doubtless, will be perplexed by the diversity of quoted opinions on the relative economics of the hingeless arch. One authority states that the hingeless is more economical than a two-hinged or a three-hinged arch; another that the three-hinged arch is the most economical; and still another that the hingeless type is more economical than the two-hinged and the three-hinged; and that the two-hinged is about as economical as the three-hinged. The whole question, however, turns largely on the rules used for allowances of metal for stresses of different kinds. Take the case, for example, of a truss continuous over two spans. Most engineers would consider that economy would be effected by making the span continuous; and yet the writer knows of a case where the spans considered singly were as strong as when considered as continuous. The ruling factor in this case was the extra allowance made under rules for the reversals of stress. Without that extra allowance the continuous girder would prove to be the most economical. Dr. Waddell finds that, if the extra effect of stress reversals is ignored, the hingeless and the two-hinged ribs will be lighter than the three-hinged ones; also, that, if the extra effect of stress reversals is allowed for, there is not much to choose as regards economy in favor of the two-hinged type over the three-hinged; also, that, if the extra percentage to be allowed for reversals is 75% instead of 50%, as some engineers specify, the weights of the two-hinged and three-hinged types will be alike. The author also considers temperature stresses, and finds that, if they are provided for without increasing the working stresses, there will be a 7% advantage for the three-hinged type; but if, on the other hand, a 20% increase of intensities of stress is allowed for the combination of the stresses due to all loadings and temperature effects, then the two-hinged type, as far as weight of metal is concerned, has the advantage over the three-hinged type. These conclusions are exhibited in Table 6, from which it is found that, both for railway and highway bridges, the hingeless arch is the heaviest, the three-hinged arch comes next, and the two-hinged is the lightest.

Problem No. 6.—The author, in a few words, describes the economies which result from a combination of the two-hinged and three-hinged types, by which the structure is three-hinged for the dead loads and two-hinged for the live loads. The relative weight of this combination,

Mr. Sales.

compared with the three-hinged structure, is estimated at 97% of the latter in the case of railway arches, and 99% in the case of highway arches. The advantages of the combination type over the three-hinged type are: greater rigidity in the moving load, and less difficulty and expense in erection.

Problem No. 7.—This part of the paper should be of great interest to Indian engineers, and the two layouts given by the author will be much appreciated should the occasion arise when an arch bridge is contemplated for a site where satisfactory anchorages cannot be found, but where it is desirable to secure the economy of such a bridge with its beauty of structure. The layout for a cantilever arch bridge indicates the lines on which designs of bridges might, with advantage, be prepared to span some of the well-known openings similar to those on Indian railways in hilly country. In such cases it is often found that the gorge abruptly descends to a great depth at a distance, say, of 100 to 150 ft. from the bank, and good rock foundations can only be found at some considerable distance below, and away from the abutments. The bridge shown by Fig. 14 seems to be an ideal type for these conditions. A layout for an arch bridge, with flanking simple-truss spans is shown on Fig. 15. It would seem that this design is not so adaptable for Indian conditions as Fig. 14.

Problem No. 8.—The author remarks that the bridge engineers of America were unable to furnish reliable data relating to the ratios of weights of metal in arch bridges and truss spans. He, however, evolves three formulas which give percentages to be applied to weights of metal in trusses of simple-truss spans in order to find the weights for arch ribs and the superimposed columns with their bracing to carry the same live loads. Results obtained from the three formulas are plotted for steam-railway, electric-railway, and highway structures as diagrams on Fig. 16. For spans of less than 100 ft. the economy of the arch bridge is not so marked, and ranges, roughly, from 75 to 100% of the simple-truss span. For spans of 300 ft., however, the percentage lies between 63 and 82. The diagram is instructive and comprehensive. It may be pointed out, however, that only the extreme limits of the percentages for 100-ft. and 1 000-ft. spans were computed, and the middle one interpolated by judgment. Diagrams of weights of metal in single-track, steam-railway arch bridges of carbon steel under seven classes of loading are given on Fig. 17, and similar information for double-track bridges on Fig. 18. Weights of metal in highway arch bridges of carbon steel with paved roadway on reinforced concrete base, and reinforced concrete sidewalks, and carrying a double-track electric railway at the middle of the deck are furnished for five widths of deck on Fig. 19. Weights of metal in single-track and double-track steam-railway arch bridges have been computed for nickel steel, and the results exhibited on the diagrams shown on Figs.

20 and 21. All these diagrams are valuable; and, although absolute accuracy cannot be depended on, for the reasons stated by the author, yet they are in most cases sufficiently so to enable the engineer to frame comparative estimates and fix the dead loads of an arch structure before preparing his final stress sheet. Mr. Sales.

David A. Molitor,* M. Am. Soc. C. E. (by letter).—In this paper the author has presented, in very well arranged form, the conclusions deduced from an extensive set of calculations bearing on the economics of steel arch superstructures. As arch bridges have grown rapidly in popular favor, during recent years, in America, chiefly for esthetic reasons, such extensive investigations are both interesting and valuable. The findings, as recorded, though not always conclusive, will at least serve as a guide in designing, and will save others many unnecessary labors. Mr. Molitor.

"The Economics of Steel Arch Bridges," of course, would include economic relations between superstructure and substructure costs in any given case, and, though not intending any adverse criticism because this phase of the subject was neglected, and necessarily so, as only generalities could be treated and the substructure always depends on special conditions, yet the paper does not cover all that is implied in its title.

The comparison between arch and simple truss superstructure weights is of limited value, because the cost ratio is generally reversed when the substructures are considered.

The fundamental problem in the economics of arch bridges consists in solving the conditions for economic span lengths for a given bridge profile comprising many spans. It might be supposed that any set of spans complying with the economic rise to span-length ratio would fulfill the requirements for a complete structure. In other words, any intermediate span would have a length of about four times its available rise. This would be true only for exceptionally favorable foundation conditions, for which the cost of piers and foundations might become relatively negligible. However, span lengths thus chosen would never result in maximum economy for end spans and for less favorable foundation conditions. Thus, the author's answer to his first problem is not conclusive, as he deals only with proportions of superstructure dimensions. His economic rise to span-length ratio, therefore, is correct only for a single span of the superstructure, as for a bridge spanning a rocky chasm where no expensive abutments are required.

Generally, however, the author's ratio must be increased until the resulting horizontal thrust is reduced to an amount where the cost of abutments attains a consistent minimum. This frequently leads to a ratio approaching 0.5, resulting in a small increase in cost of super-

* Detroit, Mich.

Mr. Molitor.

structure, which will be more than equalized by the saving thus accruing to the abutments, if it is a single span, or to the piers for an intermediate span.

Although such a high ratio is practicable, especially for stone and concrete arches, yet the shape of arch resulting therefrom is not always pleasing from the esthetic viewpoint, which so frequently interferes with economic considerations.

Hence, for all practical purposes, one might adopt the rule, that the rise-span ratio should be chosen so that it would produce the best general appearance for the least total cost of structure. This rule, probably for centuries, has received tacit acceptance without getting into print.

The author states that "in order to cover effectively the ordinary range of span lengths for arch designing, it was necessary to compute for three lengths." The writer wishes to point out that it requires more than three points to fix a curve or law, and even four points might prove inadequate for empiric laws of the kind here sought.

In solving Problem No. 2, the author's reasoning in finding the economic rib depth for braced-rib arches is not clear. He first explains that this question does not arise in the case of solid-rib arches, because the rib acts both as a beam and a strut, and then proceeds to apply the economic criterion of ordinary girders (whether solid or open-web) to the braced rib, which likewise resists bending and compression. The only difference is that open-web bracing carries shear only, while the chords or flanges receive direct stress from arch thrust. Hence, in neither case does there appear to be any justification for the assumption that the economic depth, d', is one for which the chord area, A_m, is equal to the web area, A_w (for live load only), even when the rib center line is properly located.

The suggestion of making the arch depth, at mid-span, equal to from 6 to 8% of the span length, for spandrel-braced arches (presumably three-hinged) seems to be contrary to economy; and the uniformity in chord sections, resulting from rib-depths proportional to the bending moments, is, in the writer's opinion, a decided economic advantage, both in weight and in design, which should not be disregarded.

It is strange that the author did not give more attention to the question of the economic shape of the arch center line, as even slight deviations affect the weight of superstructure very detrimentally, and any resulting increase in weight affects the cost of abutments and foundations. This, perhaps, is the most important general principle in arch economics, which has been shamefully disregarded in the past, though it more vitally affects concrete and masonry designs. Actually, the eight economic problems treated are more or less secondary in importance to the economic shape of the arch center line.

Mr. Molitor.

In his paper, "Three-Hinged Masonry Arches; Long Spans Especially Considered",* the writer showed that the center line of the arch ring should be the resultant polygon drawn for the full dead load plus one-half the uniformly distributed live load (including impact), acting over the entire span, to produce the maximum economy in material for the unsymmetric maximum and minimum live-load conditions.

The writer again discussed this economic feature in his treatise, "Kinetic Theory of Engineering Structures," on pages 305 and 323, and shows, further, on page 307, that an arch ring, formed to meet this condition, can never be truly circular or parabolic in shape. The resultant polygon becomes a circle when the superimposed load at the springing points, for semicircular rings, becomes infinite. This polygon becomes a parabola for an absolutely uniform load per foot of bridge. Neither case of loading can be realized in practice, though very flat arches closely approach the parabolic shape.

A very important result, obtained from a proper choice in the shape of the arch ring, is that all normal sections of the ring will be symmetric with respect to the axial line, or resultant polygon. A further economy in detailing will thus be accomplished, because the equal maximum bending moments (positive and negative) for each section will also have attained their smallest values for any given depth of ring.

Although this matter is most important when dealing with concrete and masonry arches, it remains a significant factor in the design of large metal arches, but may be neglected, for esthetic reasons, on small bridges.

The author is inclined to adopt the same economic rib depth for three-hinged, two-hinged, and hingeless arches, and then says that for large two-hinged arches, such as the Hell Gate Bridge, the depth should vary in proportion to the bending moment. He finally states that esthetics and rigidity should never be sacrificed for the sake of economy in weight of metal.

These several statements regarding the economic depth of arch rib, which are quite inconclusive and at variance with each other, might have been condensed into the simple rule—do as you please, or as you think best.

In considering Problem No. 3, which deals with the location of the crown-hinge of a three-hinged, spandrel-braced arch, in either the top or bottom chord, it would appear to be almost self-evident that less material would be required for the top chord hinge because the horizontal thrust is diminished by the greater rise resulting from this layout. The same circumstance would have an economic effect on the substructure or abutments. Hence, it is prudent to choose a minimum crown depth consistent with good details.

The conclusions found for Problem No. 4 are apparently correct,

* *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 61.

Mr. Molitor.

provided the bottom chord of the spandrel-braced arch is formed to meet the economic requirement for shape. The solid-rib arch is necessarily somewhat wasteful of metal, especially in the web, precisely as for simple plate girders. The braced rib having two compression chords of about equal sections will usually require more metal than the single bottom chord of the spandrel-braced arch, where the chord section is consolidated and the bending moments are carried by the spandrel bracing in direct stress.

Regarding Problem No. 5, the author's conclusions, on the relative economics of hingeless, two-hinged, and three-hinged types, are, after all, based on a somewhat meager assortment of data, despite his labors expended toward its solution. In the first place, he did not give due consideration to the special advantages accruing to each type (as a time-saving expedient), and hence the weights found are not strictly comparable. The assumption of a single value of impact for all truss members is in itself sufficient to wipe out the comparatively small weight differences sought, as the loaded lengths governing the critical stresses in a member, similarly located in each type, is materially different, which may be shown by a comparison of the influence lines for the three cases.

Under such circumstances, it is not at all surprising that he found the economics of the three types depending almost entirely on the "method of treating reversing stresses, and the importance accorded to temperature stresses." This raises the pertinent question regarding the applicability of specifications to arch bridges, when such specifications were developed for simple truss bridges only. There are numerous inherent differences between arches and trusses not covered by the ordinary bridge specifications, and any one familiar with arch designing will realize that these differences will suffice to explain the disparity between the author's conclusions and the findings of others quoted in the paper.

The writer can see no justification for treating reversing stresses according to the author's specifications for truss bridges, as the two cases of stress are produced at different times, by widely different loadings (both in position and magnitude), and different impact, and, therefore, can have no simultaneous static effect on which to base the section of any one member. Furthermore, the formulas of Launhardt and Weyrauch, based on the experiments of Woehler and Bauschinger, furnish no justification for applying the principle of fatigue to the design of any bridge members, as is best shown by referring to the experiments themselves. On this point see "Fatigue of the Material."*

Hence, all that can reasonably be required is to design each member so that it will withstand safely its maximum and minimum conditions

* "Kinetic Theory of Engineering Structures", p. 265.

of stress, one at a time, without exceeding the allowable units of stress for each condition. Ordinary prudence would require some provision for a future increase in live load and still have a structure of uniform strength throughout, relatively to the elastic limit of the material. This can be accomplished by designing all web members, or those in which the total stress is not proportional to the total loading, for such augmented live loads on the basis of appropriately higher unit stresses, chosen so that the actual unit stresses shall exceed the allowable values in a constant ratio. Mr. Molitor.

To make this clear, suppose the structure is to be made safe for 100% overload in live load only (which is extreme), then the D. L. + 2 (L. L. + *I*) stress in the chords may become, say, 1.6 times as great as for D. L. + L. L. + *I*, depending on the relative magnitude of D. L. to L. L. This would result in raising the allowable unit stress 1.6 times, or to, say, 80% of the elastic limit. Then, if the web system is designed for stresses resulting from this same overload, using unit stresses 1.6 times the allowable values, the structure will be of uniform strength throughout with respect to the elastic limit of the material, for live loads equal to twice those normally assumed. This is the writer's method of designing all structures, and puts the extra material where required to strengthen the weakest parts.

From theoretic considerations alone, the relative weights of metal, for the three types of arches, should be approximately as given by Malverd A. Howe, M. Am. Soc. C. E., quoted on page 19. However, local conditions and practical reasons frequently preclude the possibility of adopting the most economic type (the hingeless), especially for long-span bridges. Also, as pointed out before, the weight of superstructure alone, does not usually decide the economic status, so that each case will still require investigation for the conditions peculiar to the site. The rise-span ratio, also, has an important bearing on this problem, and may be a determining factor, especially for large half-through arches like the Hell Gate Bridge.

The conclusion to Problem No. 6 leaves no room for comment, as the larger variables are here eliminated and the answer becomes more or less obvious.

The cantilever arch bridge, which received attention in Problem No. 7, represents a type which has not been fully appreciated, though its application is somewhat limited; but, where the profile and foundation conditions fit such a layout, there is certainly every argument in favor of a two-hinged cantilever arch. The stress calculations offer no special difficulties, provided influence lines are used. Any other method of calculation might prove hazardous on account of the complicated load positions for critical stresses. The author's findings relative to the proportions of span lengths are very valuable.

Mr. Molitor.

It should be mentioned that a three-hinged cantilever combination is usually undesirable, and may possibly result in unstable forms.

The final weight curves, presented under Problem No. 8, for arch superstructures, constitute a valuable contribution to our meager knowledge of this subject. These curves will at least save the labor of a preliminary weight calculation in most cases, thus permitting the designer to proceed at once to a fairly comprehensive stress analysis.

Exception might be taken to the author's reasoning applied in finding the maximum possible span length for three-hinged arches. He makes W_a infinite and finds the corresponding span length, $l = 3\,540$ ft.; but W_a is the weight of metal per linear foot of span, and when this is infinite, the dead-load stresses in all principal members become infinite. This is evidently not an answer to the question of what span length will result in such a dead load that the unit dead-load stresses in the members will not exceed the maximum allowable values normally assigned for total dead plus live load. Even this span length, if known, would exceed all limits of usefulness, as a bridge of this span could carry no live load.

The other assumption: "that the said limit is reached when it takes $4\frac{1}{2}$ lb. of metal to carry 1 lb. of live load," leads to a very much shorter span, which the author estimates at about 2 000 ft., though this would lead to an enormously expensive bridge.

The author's opinion "that if more were known generally concerning the weights of metal in steel-arch structures, their economic functions, and their comparative economy in respect to the corresponding simple-truss structures, the esthetic type would be adopted more frequently, in spite of the greater labor it involves in both designing and construction," does not appear as a real excuse for not adopting the arch type whenever that type is warranted.

In the first place, the weight data referred to are merely a convenience and, if unknown, do not constitute a deterrent factor to any one of earnest endeavor, as is manifested by the numerous steel arches designed in the past. Then, also, the greater labor involved in designing arches depends very much on an appropriate choice of the methods used, and on the competency and experience of the designer; but the greatest obstacle to popularizing arches is, perhaps, the personal bias frequently expressed against this type by bridge engineers themselves, owing largely to a limited familiarity with this more specialized branch of bridge engineering.

In fact, the author himself has frequently expressed his views adversely to all structures involving redundancy, and his present effort may be accepted to indicate that he is gradually changing his views on this subject.

It is the writer's opinion that arches of steel, concrete, or reinforced concrete, should receive the first consideration, whenever the profile,

Mr. Molitor.

site, and foundation conditions warrant their construction, regardless of any lack of preliminary weight data, or the slightly greater labor involved in their design. In thus minimizing the difficulties and expenses attending such designs, he is actuated by the results of his own experience, with the general comprehensive methods (mostly graphic), presented and advocated in his recent treatise previously referred to, and which can be more fully appreciated by any one who will approach the subject with an open mind. That many bridge engineers have neglected to post themselves, is manifested by articles in current literature and professional papers, where cumbersome and obsolete methods are often presented as new and novel.

The author deserves much credit for his present contribution on arch economics, a subject about which little has been published. The paper points the way to accomplishing more in this direction, and emphasizes the importance which the designer should give to economic studies.

Mr. Stoney.

EDWARD W. STONEY,* ESQ. (by letter).—The writer has read with great interest this most valuable paper, but regrets that he is unable to add usefully to its discussion, as Indian engineers have had no occasion to consider, or opportunities to use, metal arch bridges.

The many large and long railway bridges which have been built in India are generally over sand and silt river beds of great depth, and require foundations sunk to from 100 to 150 ft. below these beds; in fact, at the "Lower Ganges" Hardinge Bridge, recently built under the supervision of Sir Robert R. Gales, the foundations are from 180 to 185 ft. below the river bed.

The banks of these rivers, also, are often but little above the river beds, so that arches would be quite unsuitable because as much waterway as possible has to be provided, without unnecessarily raising the approaches.

The writer thinks that the economics and costs of maintaining girder and arch metal bridges of similar span might be given by American engineers who have had to do with both. The maintenance of long-span girder bridges with intricate systems of bracing causes considerable expense, trouble, and anxiety to railway men.

The writer desires to invite the attention of American engineers to a rather remarkable book† by the late Sir B. Baker, Hon. M. Am. Soc. C. E., entitled "Beams, Columns, and Arches", in which the author discusses metal arches, gives simple formulas for computing their approximate dimensions, and compares the results obtained thereby with the elaborate calculations made by the late Capt. J. B. Eads, M. Am. Soc. C. E., for the St. Louis steel arch bridge.

* Coonoor, Madras Presidency, India.

† Published by E. F. Spon, London, 1870.

Mr. Stoney.

The writer would also call attention to the paper* entitled "Curve of Equilibrium for a Rigid Arch under Vertical Forces", by Mr. George Fuller. In this paper Mr. Fuller shows how this curve may be derived graphically, at once, from the bending moment diagram produced by these forces.

Mr. Frankland.

F. H. FRANKLAND,† M. AM. SOC. C. E. (by letter).—Owing to the wide divergence of opinion regarding certain economic functions of arch design, which is shown by the writings of Howe, Merriman and Jacoby, and others, and well illustrated by the tabulation of the principal steel arches of the world given in the discussion by Charles E. Fowler, M. Am. Soc. C. E., the writer considers the author's investigations concerning the economics of steel arches of very great value to the Profession, especially as they constitute the first systematic attempt at the solution, so far as he is aware, of the primary problems pertaining to arch design. Mr. Fowler's discussion undoubtedly adds materially to the value of the paper, and he is to be commended for the energy and hard work shown in compiling the table of bridges. It is to be noted that Mr. Fowler has omitted from his table any mention of the three arches calculated by the late C. R. Grimm, M. Am. Soc. C. E., and illustrated in his paper "The Arch Principle in Engineering and Esthetic Aspects, and its Application to Long Spans".‡

Mr. Grimm stated that his comparison of five designs for one crossing, where the central opening was the same, *viz.* 1 800 ft., showed the total metal in the superstructures to be as follows:

1. Cantilever bridge.....	70 000 tons
2. Suspension bridge.....	60 000 "
3.—Arch bridge (1 800-ft. central span flanked by two crescent arches)..	53 000 "
4.—Arch bridge (1 800-ft. central span flanked by two tied arches).....	45 000 "
5.—Arch bridge (1 000-ft. arch span at center, flanked by two cantilever truss spans, each projecting 400 ft. beyond main piers).....	45 000 "

Mr. Grimm's computations and speculations are especially interesting, from the point of view of economics. His computations, however, applied to the superstructure only, and it must be borne in mind that a consideration of the substructure design might have caused the total costs of his five cases to assume a different order in economy.

The author's well-known characteristic of "stick-to-it-ive-ness" is illustrated by the subject of his paper. In 1897 he wrote § as follows:

* *Minutes of Proceedings*, Inst. C. E., Vol. XL (1874–75), p. 143.

† New York City.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 233.

§ "De Pontibus."

Mr. Frankland.

"In concluding this chapter, the author desires to call attention to the fact that there is still a great deal to be learned about the designing of arches; and to suggest that some professor of civil engineering, who is well posted on bridge designing and who has time to spare, could spend several months to the great advantage of the engineering profession in determining the proper relations of span length, rise, arch depth, width between exterior arches, etc., for the various styles of arch, and in ascertaining the relative economics of the latter."

and, in 1915, he stated:*

"For many years the author has been endeavoring to establish some approximate relation between the weights of metal per lineal foot for trusses and laterals of arch bridges and those for the corresponding simple truss bridges; but has met with very little success. He once submitted the question to his brother bridge specialists of America, but they were unable to throw any light upon the subject, because their opportunities to design and build arch bridges had been few and far between, and because the ratio of rise to depth has a great effect upon the weight of metal in an arch. Of course, there is for any span length some economic value of that ratio; but it is not yet known, and it probably varies more or less not only with the span but also with the type of construction. The only practicable method of determining the original question would be to settle first that of the economic ratios of rise to span, design a few arch bridges with the said ratios, and make the comparison. This would have to be done for the solid-rib, the braced-rib, and the spandrel-braced types to make the job complete, adopting for the first set of curves the three-hinged type, and afterward modifying the results for the other three types of hinging. It is evident that the amount of work involved in such an investigation would be immense. It should be done by an experienced bridge designer, as the results would be worthless if obtained by any other investigator. The author suggests that one of his younger brother-specialists undertake the investigation."

Between the writing of "De Pontibus" and "Bridge Engineering", the author made several systematic attempts to induce other engineers to undertake such an investigation, but without success. He even went so far as to attempt to induce some technical institution to agree to give doctors' degrees to two bridge specialists who would do the work jointly, but that failed to materialize.

Generally speaking, practical as well as economic considerations point to the use of two-hinged, spandrel-braced arches as probably the best type to use for railway traffic, provided that the natural abutments permit of sufficient rise, which is often the case where the selection of an arch design is a logical solution of a particular problem. The spandrel-braced type of arch is less subject to vibration and is more rigid than any other type; and it is peculiarly suited to easy erection by the cantilever method, as was done in the case of the author's 425-ft. span

* "Bridge Engineering."

Mr. Frankland.

arch bridge on the Canadian Northern Pacific Railway over the Fraser River at Lytton, B. C. The inclination of the arches to the vertical contributes greatly to rigidity, this being particularly noticeable in the Fraser River structure.

In connection with the erection of two-hinged arch bridges it is customary nowadays to adopt an expedient to obviate the usual necessity of introducing the calculated stresses due to dead load and temperature in the top chords at the center by the use of powerful jacks, or other means, before the final point can be riveted up. The expedient referred to consists in assuming that the dead load and temperature stresses in the top chord at a certain selected temperature are zero, the arch acting as a three-hinged structure under these conditions, namely, with all dead load in place and at the selected temperature. The span, therefore, is erected as a three-hinged structure, all dead load, or its equivalent, including the decking, being placed thereon, and at a time when the actual temperature is as close as possible to that previously selected. The center panel of the top chord is then easily inserted and riveted up. This expedient was used by Ralph Modjeski, M. Am. Soc. C. E., for the erection of the two-hinged, spandrel-braced, arch bridge for the Oregon Trunk Railway in the Crooked River Canyon, and by Gustav Lindenthal, M. Am. Soc. C. E., for the erection of the Hell Gate Arch.

It might not be out of place to mention here the frequent error which some make in considering arches of the Hell Gate and Detroit-Superior type as being of the so-called "open-web" type. Although the two instances cited have curved top chords, principally for the sake of appearance, they are both essentially "spandrel-braced" arches, which usually have straight horizontal top chords. Both the Hell Gate and Detroit-Superior bridges have special provision in their design to guarantee that the wind stresses would be taken to the lower chords and would not be transmitted to the top chord. The reason for this is obvious, and is symptomatic of the spandrel-braced as compared with the open-web type, where the main stresses are usually divided between top and bottom chords and are transmitted equally thereby to the skewback hinges.

It appears to the writer somewhat surprising that the cantilever type of steel arch has not been used more frequently in America, although its applicability to certain conditions has long been recognized in Europe. As the balancing of the moments about the hinges very materially reduces the horizontal reactions at the abutments, it is possible in many instances to effect a material economy by the utilization of this type. The two best-known examples of the cantilever arch are the Viaur Viaduct, in France, and the Rio Grande Bridge, on the Pacific Railway, in Costa Rica. This type of construction is particularly easy and economical of erection by the cantilever method.

Mr. Frankland.

The economic gain of the arch over the truss bridge is greater in highway than in railway bridges, for several reasons, chief among which is the smaller ratio of impact allowance to total dead load. This is the principal reason why three-hinged arches are more suited to highway than to railway structures.

It would seem that the use of nickel steel in arches, where the ratio of rise to span is small, would materially affect the economics, since the reduction of horizontal thrusts at the abutments, due to reduced dead load, would be considerable. This, however, is a question of balancing the increased pound price of the alloy steel against the saving in weight of steel, plus the increased economy of the abutments.

The writer feels impelled to mention one consideration governing the economics of arch bridge design which the author has not noted in his paper, perhaps because it relates to substructure design. The important bearing on economics, in this connection, is, as to whether or not it is possible to place all the piers of an arch bridge on the same kind of foundations, rock or pile. It is well known, of course, how extremely unwise it is to found one abutment on rock and the other on piles. The consideration of this condition has quite recently been experienced by the writer in his practice, the instance being of a certain concrete arch bridge, with a bascule span at approximately the mid-point, thereby dividing the structure into what are really two arch bridges. The rock outcrops near one end but dips toward the other end, so that the depth to rock makes it impracticable to found the entire structure on rock. It will be necessary in this case to carry the piers to rock for one-half of the structure and to carry the piers of the other half on piles, although by doing this the deepest abutment on rock and the shallowest abutment on piles are, considered alone, uneconomic.

The writer ventures to express the opinion that there have been numerous occasions when an arch design would have been more economical than the truss design which has been adopted. Perhaps this has sometimes been caused by the "bogey" of the computations involved in the arch design. He thinks that this bogey would be readily dispelled if bridge engineers were to make more common use of the graphic instead of the analytic method of finding the stresses. The graphic method is not only shorter, but is self-checking, and it enables the computer to follow each step very clearly. If any should doubt this assertion, let him try both methods on the same problem and become convinced. Means of popularizing the building of arches in many places similar to those where truss and cantilever spans have been adopted, such as papers like the author's, should be strongly commended.

Steel arch bridges are generally suitable only for those sites where natural abutments exist, or where the surroundings are such as to call for special attention to esthetics. The latter is probably one of the

Mr. Frankland.

principal reasons why a larger percentage of bridges in Europe are of the arch type, although it is undoubtedly true that, as a rule, bridge sites in Europe offer greater natural advantages for arch design than have so far existed in the Western Hemisphere. However, with the continued development of North and South America, rougher terrain will be brought under the hand of civilization, and this difference will, to a great extent, eventually disappear. Consequently, it seems to the writer that an increasingly large number of arch bridges will be built in the United States.

The application of economic theories of arch design is necessarily restricted by the local conditions at the site, which invariably govern the ratio of rise to span, and the proper determination of the true economics calls for more judgment on the part of the designing engineer than with any other type of bridge. By "judgment" the writer means the developed engineering judgment which comes from long practical experience in problems relating to the choice of location and the right interpretation of existing conditions.

Some bridge engineers seem to be imbued with the idea that, if natural rock abutments do not exist, it is not worth while to consider the feasibility of an arch for a particular site. In refutation of this erroneous idea, the writer begs to draw attention to the concrete arch structure having the longest span in the world, the hingeless arch of 327-ft. span over the Tiber at Rome. As is well known, the abutments for this bridge are founded on alluvial silt. In this case, other conditions than those of economics were predominant.

Particularly noticeable in the paper is the neatness of the method used in determining the economic ratios given in Fig. 13 by means of Fig. 12, as is also the usefulness of the equations given on page 27.

Mr. Waddell.

J. A. L. Waddell,* M. Am. Soc. C. E. (by letter).—The hearty thanks of the writer are tendered to all who have done him the honor of discussing his paper, and especially to Mr. Fowler for the many weeks of hard work which he devoted to the compilation of the table of data (Table 8, Plate I) concerning one hundred long-span arch bridges. It is a matter of regret that there were not more contributors to the discussion, especially as so many were asked individually to take part therein. The reason for this, undoubtedly, is the very limited amount of knowledge in the Profession, previous to the publication of the paper, concerning the economic questions treated and the weights of metal in steel arch structures.

Of the eight primary economic problems set originally for solution in this paper, six were solved by making special designs and subsequent computations of weights of steel, thus settling, for these cases, beyond

* New York City.

all doubt whatsoever, the question of economy of metal; but the other two were based on certain semi-rational, semi-empirical formulas that give the approximate weights of carbon steel in braced-arch ribs. In these problems the validity of the conclusions reached in the investigation was legitimately subject to question; and that question was raised by Mr. Fowler, who based his criticism mainly on averages of percentage functions taken from the very complete table of long-span arch bridges which he compiled. Mr. Waddell.

He showed that the writer's economic ratios of rise to span length for the different types of arches agree fairly well with the superior limits found. Possibly this is merely a fortunate coincidence, because designers are often prevented by ruling conditions from adopting what they might deem to be economic ratios of rise to span length; but, on the other hand, it might be considered by some as corroborative evidence of correctness.

In the other of the two problems, however, *viz.*, the economic depth of arch ribs, there is seldom any condition precedent which should prevent a designer from adopting approximately the most economic depth; and hence averages of depths of ribs of existing arches should show what the general impression of designers has been concerning this question of economics. Mr. Fowler computed these averages and found results considerably lower than those which the writer determined by the use of his before-mentioned formulas for weights of arch ribs. From individual judgment, based on these averages, and on the writer's findings as well, Mr. Fowler determined certain compromise economic depths varying from 2.5% for plate-girder arches to 4.0% (and in one case 4.3%) for lattice-ribbed arches. Instead of 4.0%, the writer had found 7.5% as the most economic depth, but had pointed out that a material variation can be made therefrom without greatly increasing the weight of metal in the arch, also that "any variation that is likely to be adopted will be a reduction; because, generally, the appearance of the arch is improved by making its rib depth smaller than the economic one." Computations made from Fig. 6 show that, on the basis of the writer's original investigation, decreasing the ratios of depth of arch rib to span length would give certain percentage increases in weights of metal in the ribs. These are shown in Table 12.

In view of the preceding, it is evident that the writer, when proportioning an arch rib, would probably have adopted, for esthetic reasons, a depth equal to about 6% of the span length, notwithstanding the fact that he had shown the theoretically economic depth to be 7½% thereof.

When Mr. Fowler challenged the findings of the investigation on this point, the writer stated that he would prepare four designs and estimates of weight for the ribs of a 500-ft. span, double-track railway, three-hinged arch bridge, so as to plot a curve indicating the actual

Mr. Waddell.

economic depth; and he has since done so, the depths chosen for trial being 3, 5, 7, and 9% of the span length. These layouts are shown to scale on Fig. 32. The computations of stresses were made by the writer's Assistant Engineer, Mr. Francis De Schauensee, using the modern method of influence lines, but, in accordance with the writer's suggestion, checking one set of figures by the old established system of unit panel loadings. Although the check, of course, was not exact, it was sufficiently accurate for all practical purposes.

TABLE 12.

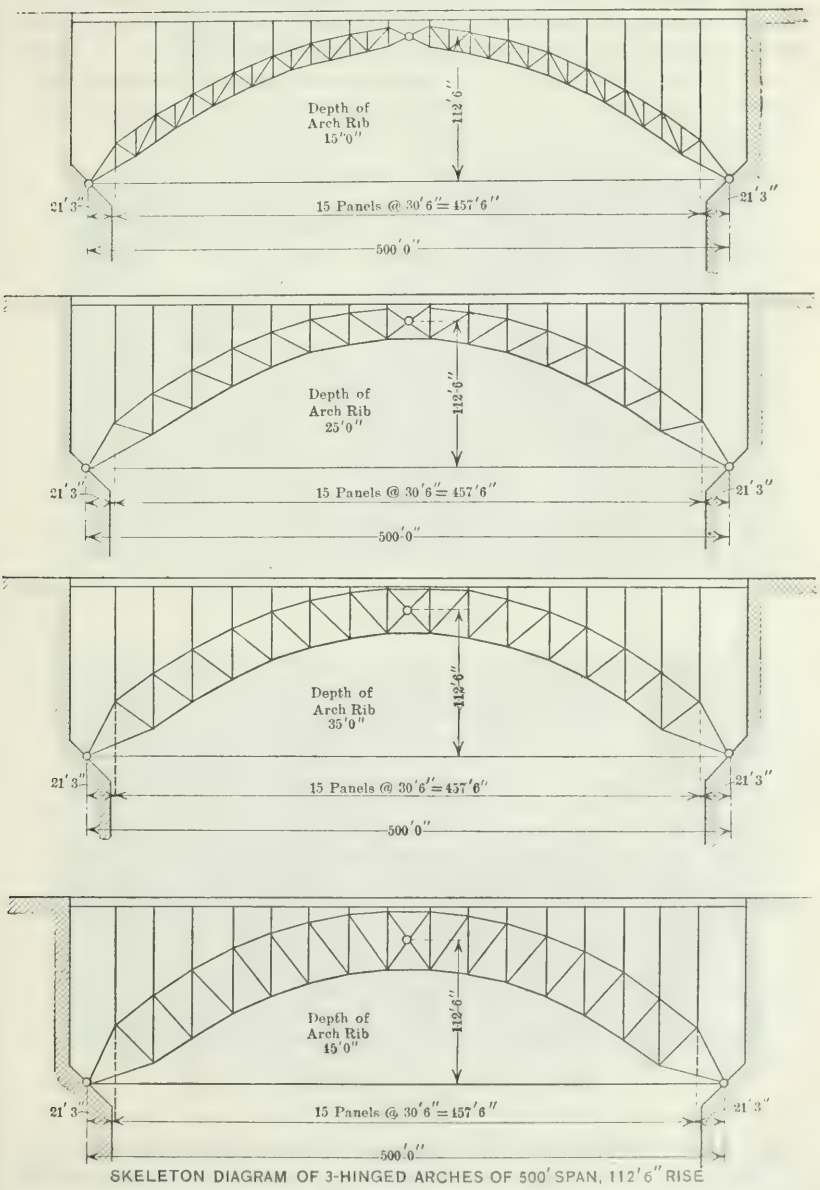
Ratio of depth of arch rib to span length.	Percentage increase in weight of metal above minimum in arch rib.	Ratio of depth of arch rib to span length.	Percentage increase in weight of metal above minimum in arch rib.
0.75	0.00	0.55	2.75
0.70	0.20	0.50	4.20
0.65	0.65	0.45	6.80
0.60	1.30	0.40	10.50

With these stress sheets as a basis, and using the designing specifications of "Bridge Engineering", the writer computed the sectional areas of all the main rib members and measured their lengths from the diagrams, reducing properly those of the web members to allow for their failure to reach the center lines of the chords. Then he multiplied each tabulated sectional area by the length of the piece, added the results, and multiplied the sum by 3.4, the product representing the weights of the main members, exclusive of the details. All the latter, in which the weights thereof are approximately alike for the four cases, such as stay-plates, connecting plates, diaphragms, hinges, and rivet heads, were omitted, but the weights of the splices and the lacing angles were computed and added in. The four quantities thus found were used as "comparing weights" to settle the economic question which was being investigated.

These "comparing weights" were plotted on Fig. 33, and a curve was drawn through the located points. Considerable thought was given to the drafting of this curve between the limits of 5% and 3% depths, owing to the abnormality of the latter layout. It would have been somewhat more accurate to have computed the weight for a 4% depth, but the time and expense involved thereby did not appear to be justified. From this curve, it will be seen that the economic depth of the rib for this span and for railway loading is $6\frac{1}{2}\%$ of the span length, instead of $7\frac{1}{2}\%$ thereof, as previously determined.

Recognizing that the making of a corresponding set of calculations for a highway bridge would be a comparatively simple matter, all

Mr. Waddell.



SKELETON DIAGRAM OF 3-HINGED ARCHES OF 500' SPAN, 112'6" RISE

FIG. 32.

Mr. Waddell.

stresses being found by slide-rule, the writer concluded to investigate the correctness of his previous finding, *viz.*, that the economic rib depths are the same for both railway and highway bridges. Some *à priori* reasoning, based on the manner of variation of the governing adjusted total stresses, had led him to the conclusion that these eco-

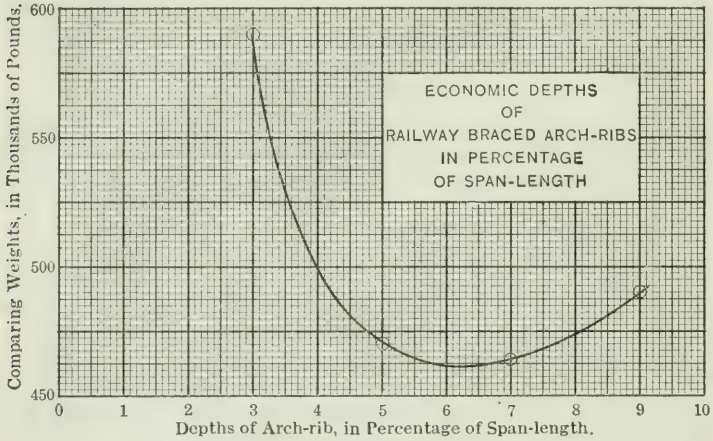


FIG. 33.

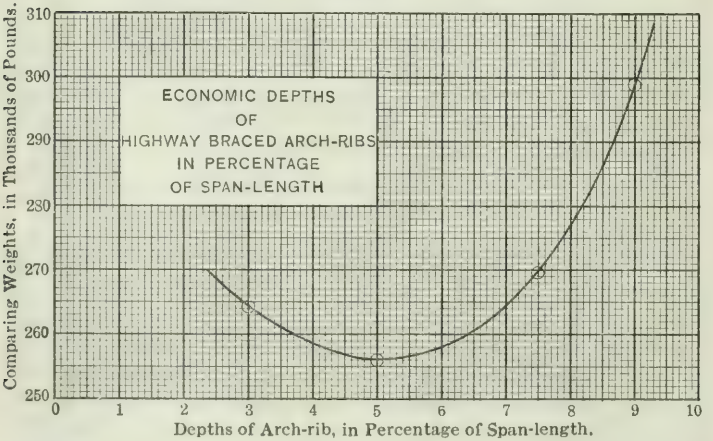


FIG. 34.

nomic depths would be less for highway arches than for railway arches, the factor of main influence being stress reversion in chords, which influence, of course, augments as the ratio of live load plus impact to total load increases.

Mr. Waddell.

Estimates of "comparing weights" were made as before for depths of 3, 5, 7, and 9% of the span length, and the results were plotted on Fig. 34, from which it will be seen that the economic depth for this case is 5% of the span length.

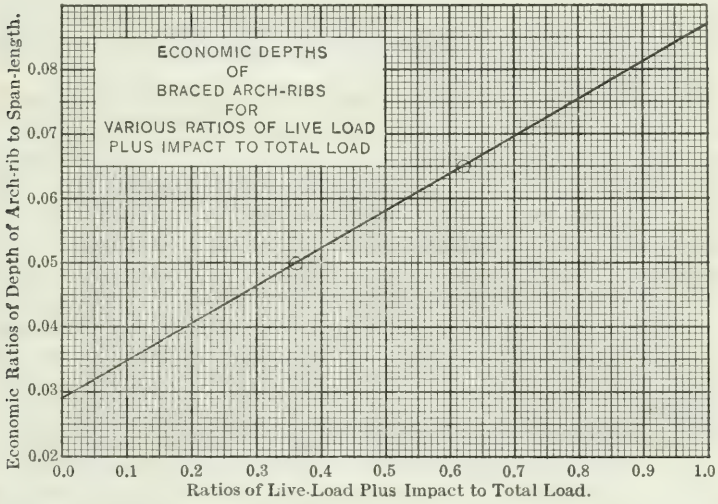


FIG. 35.

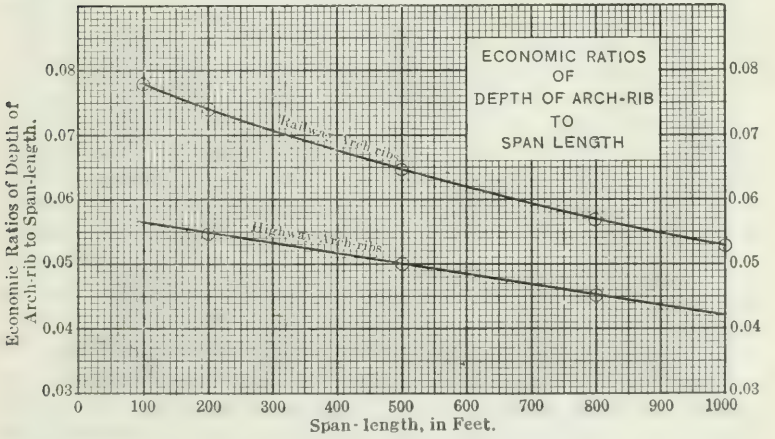


FIG. 36.

These two sets of figures permitted the determination of the manner of variation of the economic depth with the ratio of live load plus impact to total load, as shown on Fig. 35. The two points enclosed in circles on that diagram indicate the economic-depth ratios correspond-

Mr. Waddell.

ing to the ratios of live load plus impact to total load, in the case of railway and highway arch spans of 500 ft. By joining these points with a right line and extending this line in both directions, we obtain the data from which, by using Figs. 18 and 19, the diagram in Fig. 36 has been prepared. That diagram shows that the economic depth ratio for railway arch ribs varies from 0.078 for 100-ft. spans to 0.053 for 1 000-ft. spans; and for highway arch ribs from 0.056 for 100-ft. spans to 0.042 for 1 000-ft. spans.

These depths are greatly in excess of the 0.04 ratio established by Mr. Fowler; and the investigation proves that, in endeavoring to ascertain such important functions as economics, it is not safe to base one's conclusions on averages of professional practice. The writer feels greatly indebted to Mr. Fowler, not only for the good work which he has done in compiling his table of information concerning existing arch bridges and arch-bridge designs, but also for having challenged the writer's finding for economic depth of arch ribs. For railway arches the economic ratio first found proves to be all right for short spans, though too great for long ones; but the deduction made, to the effect that the economic-depth ratios are the same for both railway and highway bridges, is wrong. The reason for the occurrence of the error is that the weight formulas for arch bridges, which were adopted as a basis for the investigation, were of a too empirical character to warrant their use for the economic-rib-depth study, although sufficiently accurate for that of the economic ratio of rise to span length. The probable explanation of engineers' failures in the past to use economic depths when designing arch ribs is that they did not pay much attention to the question of economy when making their determinations.

As for the question of esthetics, the writer is of the opinion that a rib depth of 6% is more pleasing to the eye than a smaller one, and that such extremes as 3% or even 4% are unsightly. If a designer prefers the appearance obtained by using a 5% depth, he can generally obtain it in railway structures by the addition of 2 or 3% to the minimum weight of metal—certainly not a very serious extravagance.

The eight diagrams of stresses, sectional areas, and lengths of members, prepared in making this supplementary investigation, are not included in this résumé, but may be found in the files of the Society.

Mr. Fowler claims that greater ratios of rise to span length are necessary for heavy arch bridges than for light ones, because of the element of rigidity, and, therefore, he questions somewhat the writer's findings; but, concerning this claim, two facts should be borne in mind:

First.—The writer's investigations were made for economic depths, and, therefore, not necessarily for the best depths to adopt for all cases and all conditions; and

Second.—The economic ratios of rise to span length found by the writer are never small, varying as they do from 0.20 to 0.28 for solid-

rib structures, being 0.25 for spandrel-braced structures, and lying between 0.225 and 0.38 for braced-rib structures. Mr. Fowler's table shows, for structures of these three classes, respectively, the following averages: 0.18, 0.197, and 0.202, as against 0.24, 0.25, and 0.3 by the writer. Mr. Fowler's corresponding recommended averages are, respectively, 0.22, 0.254, and 0.272. Mr. Waddell.

From this it is seen, that by taking averages for these three groups of averages, *viz.*, 0.193, 0.263, and 0.249, the writer's economic ratios of rise to span length, on the average, are 36% greater than those hitherto adopted by the Profession and 6% greater than those recommended by Mr. Fowler.

The reason that, in truss bridges, light structures, for economy of metal, require smaller truss depths than heavy ones is because of the greater values of $\frac{l}{r}$ in the web members; but, in arch bridges, this condition is cared for by varying the ratio of depth of arch ring to span length, and not by changing the ratio of rise to span length.

Mr. Thomson's suggestion that, in very heavy steel bridges, box sections for chords be used, is an excellent one, and the writer for years has been advocating this innovation. In making the plans for the Fratt Bridge, over the Missouri River, at Kansas City, he adopted the closed section; but the innovation was too drastic for the manufacturer, who was also the erector of the structure, and the plans were modified to agree with current practice, in order to comply with his wishes.

Mr. Farr asks why the writer, in Tables 4 and 6, gives the weight of metal per linear foot "per side" for ribs and lateral system, while in Table 5, he gives it in pounds per linear foot of structure. The answer is that, throughout the paper, comparisons were made by excluding all weights common to the structures compared, and hence the floor-system weights were generally omitted, and one side of the structure only was considered. In Table 5, however, it was desired to show the variations in the total weights of metal per linear foot in the three types of construction, consequently, that of the floor system was included in the totals.

When Mr. Farr says: "The economy in the weight of metal in the arch over the simple truss appears, from the information in the paper, to commence at spans of more than 1 000 ft., etc.", and "As it is only in the larger spans of more than 1 000 ft. that any economy in weight is shown over even a simple truss, and as the span at which the arch will come out lighter in weight than the continuous truss will be very much more than 1 000 ft., etc.," it is evident that he made a mistake in reading Fig. 16; for, in railroad bridges, the economy of the arch is shown to begin at spans of about 100 ft. and not at 1 000 ft., as he claims, and, in highway structures, the arch bridge is indicated as always being more economic of metal than the truss bridge. There is

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no good reason for this misinterpretation, because the title of Fig. 16 reads "Percentages to Apply to Weights of Metal in Trusses of Simple-Truss Spans in Order to Find the Weights for Arch Ribs and the Superimposed Columns with Their Bracing to Carry the Same Live Loads"; and there is no percentage recorded that is as great as 100.

Answering Mr. Van Cleve's remark that "To place a field splice across the crown-hinge would undoubtedly be possible, but, in the speaker's opinion, would rarely be justifiable, owing to the quantity of splice metal required, and the expense of the field drilling", the writer would reply that this expedient has already been adopted, and without much trouble or expense. If the filling-in is done at an average temperature, the field work involved is very simple, but, at any other temperature, some special apparatus is required. The good results gained by the insertion of the additional members are worth many times the extra expense involved.

Answering Mr. Sales' remark that "Nevertheless, it is to be hoped that, judging by past experience, the author will find time to write a treatise on arch bridges, and include in it all the diagrams and preliminary work for which space could not be found in his paper", the writer would state that, some time ago, he arranged to do almost that very thing by writing jointly with a young engineer, who is expert on difficult mathematical work, a complete treatise on "The Designing of Steel Arch Bridges", the writer taking the chapters dealing with the economics, the specifications, and all the practical features, and his associate handling those involving the mathematics. The work had progressed only so far as the laying out of the list of chapters and the determination of the character of the contents of each, when, after thinking the matter over, the young man decided that he would not undertake the contemplated enterprise; hence it was abandoned. As the writer has neither the time nor the inclination to do, unaided, all the work on such a book, it will never be written by him, but must be left to some younger man, possibly working in conjunction with an experienced bridge designer and builder.

Mr. Sales asks what the writer means by "lateral system." The term includes all the bracing between adjacent trusses, ribs, or girders, whether lying in horizontal, vertical, or inclined planes; but excludes the cross-girders, which pertain to the "floor system." Mr. Sales is correct in his surmise that the writer assumed that "the weight of the lateral system plus rail-girders plus cross-girders will offset that in the bracing of the floor system, plus that of the rib flanges of the corresponding arch structure, plus that of the other diagonals between the arch ribs"; but the writer had indicated clearly that the weight of the vertical sway bracing between opposite columns was included with the weight of the said columns and the arch ribs, hence the word "other" excluded the diagonals of the vertical sway bracing.

Mr. Waddell.

The section in an arch rib will not often be "symmetrical about the neutral axis", for the areas of the chord members will generally be in direct proportion to the governing stress combinations. The contour of the arch rib usually cannot "be placed so as to give symmetrical rib sections"; but, for uniformly distributed loads, the axis (or line of apparent symmetry) is a parabola, while in other cases the curve should correspond to the equilibrium polygon for dead load plus one-half the live load.

Mr. Sales asks "what the author has to say" on the subject of the simultaneous existence of economy and esthetics in bridge design. More than two decades ago, the writer expressed himself thus:*

"The author is a firm believer in the principle that true economy, engineering excellence of construction, and the best architectural effect will almost invariably be found to accompany each other, and be inseparable in the designing of any bridge. Moreover, any bridge built with due consideration for, first, efficiency, second, appearance, and third, economy, will be satisfactory and gratifying to not only the trained expert, but also to the general engineer and the railroad man, and even to the public; because when an observer notes that in such a structure all the engineering requirements are properly provided for, that there is no evident waste of material, and that all due advantage has been taken of the conditions to render the bridge sightly and in harmony with its surroundings, his eye will of necessity be pleased, and his inherent sense of fitness will cause him to regard the structure with a feeling of pleasure."

Mr. Sales asks how the writer "would design the ordinary through type of truss span so that the top lateral bracing could be arranged to assist the top chords to the utmost extent by taking a part of the compression to which these members are subjected." The aiding of the top chords by the upper lateral struts is not a desirable feature, but, unfortunately, to a certain extent, it is unavoidable. In American practice such aid is reduced to a minimum by swinging the span clear from the falsework before making the final connection for each diagonal of the upper lateral system, thus preventing these diagonals from aiding at all in carrying the dead load. If it is considered advisable to reduce the amount of the aid which the diagonals give in resisting the live-load stresses in the chords, it can be done by putting a draw in them, or, in other words, by punching the connecting holes so as to shorten slightly the effective length of the piece and stretch it during erection by the use of drift-pins.

Answering Mr. Sales' question in respect to Problem No. 4, the comparisons were made with three-hinged arches; but the writer does not believe that the results would be any different for two-hinged arches. Hingeless construction for the spandrel-braced arch, although feasible under certain conditions, as far as the writer knows, has never

* In his little book "De Pontibus", p. 47.

Mr. Waddell.

been attempted or even considered. He can see no possible advantage in adopting it.

The writer is of the same opinion as Mr. Sales concerning the economies of continuous trusses, *viz.*, that the economy claimed by some engineers is due simply to an improper ignoring of the effect of reversing stresses. He has had in mind lately the solution of this question in the only satisfactory way possible, namely, the proportioning of two continuous and two non-continuous trusses for the same layout and the same live load under different specifications in respect to the treatment of reversing stresses. Owing to the fact that the calculation of stresses in long-span, continuous trusses is tedious, complicated, and expensive, the writer thought to shorten greatly the labor involved in his proposed economic investigation by procuring from a brother engineer the live-load stresses, dead-load stresses, weights of metal, etc., for a large two-span structure with continuous trusses, which that engineer had lately built; but the favor was refused. Some day, however, if the writer lives long enough, he will solve this and two other economic problems; because these three would about complete the solution by him of all the important economic questions that occur in the specialty of bridge designing.

In respect to the diagram on Fig. 16, concerning which Mr. Sales states that the writer computed the percentages for only the 100-ft. and 1 000-ft. spans and interpolated for all intermediate spans, there is a misunderstanding. The lines for both railway and highway structures were determined for a number of spans. It was the intermediate line for electric railway structures that was interpolated, owing to the fact that the writer has not built many electric railway bridges that do not carry also highway loads.

The writer certainly appreciates highly the trouble which Professor Jacoby took in preparing so much valuable information concerning metallic arch bridges to supplement his paper.

Mr. Molitor must have overlooked the last six lines of the penultimate paragraph of the paper, which read thus:

"Again, the comparison of the costs of arch superstructures and truss superstructures alone is not of much importance; for an economic investigation, to be of any value, must include both substructure and superstructure; and the costs of the former are likely to be very different in arch designs and simple-truss designs for any crossing."

It is impracticable to take the substructure into account when making general economic investigations for arch bridges, because no two case thereof are alike. In case of a succession of long spans in which there is a choice of rise, a tentative layout should be made, using the anticipated economic ratio of rise to span length when both substructure and superstructure costs are considered, then the cost of

piers and spans should be computed, the uneconomic effect of departing from the established economic ratio of rise to span length given in Figs. 1 to 4, inclusive, being used to calculate the weights of metal from those diagrammed in Figs. 17 to 21, inclusive. Next, this work should be repeated for a slightly greater ratio of rise to span length, and then for a slightly smaller ratio thereof. These three sets of computations would probably determine the best ratio; but, if not, still another set of calculations would be necessary. Mr. Waddell.

Mr. Molitor criticizes the writer's statement that "in order to cover effectively the ordinary range of span lengths for arch designing, it was necessary to compute for three lengths", saying that "it requires more than three points to fix a curve or law." In reply, the writer would state that the results of the calculations on these span lengths were not used to plot curves, but merely to show how the economics varied for different span lengths. An ample number of points was used in locating all the curves given in the paper.

Concerning Mr. Molitor's remarks on Problem No. 2, the questions he brings up are fully treated in the résumé on Mr. Fowler's discussion; but the writer would also remark that the fundamental principle of the method he originally used in determining the economic ratio of depth of rib to span length was logical, although not quite accurate enough, as was shown by the subsequently prepared, correct, detailed calculations, given previously in this closure.

Mr. Molitor thinks that "it is strange that the author did not give more attention to the economic shape of the arch center line." The reason therefor is that this economic question had already been solved satisfactorily by other investigators, including Mr. Molitor himself, the scope of the paper being only economic problems hitherto unsolved or incorrectly solved. Reference was made to this matter at one place in the paper, where it was stated that, if the dead load is practically uniform across the span, the curve should be a parabola; and that, otherwise, it should correspond to the equilibrium polygon for dead load plus half the live load.

The assumption by the writer of a single value for impact for all arch spans of the same length and subject to the same live load caused no error of any importance in the economic deductions, for the small resultant errors in the several spans compared were practically alike. Without having made a number of assumptions similar to this in unimportance, it would have been impracticable for the writer to undertake the heavy load of work involved by the entire economic investigation.

That there should be a difference in arch bridges and truss bridges, as regards the treatment of the effects of reversing stresses, the writer is not disposed to deny; for he once was compelled to design a 425-ft.

Mr. Waddell.

arch-span railroad bridge for British Columbia according to the awkward and, in many respects, absurd specifications for railroad bridges issued as standard for Canada by the Dominion Government. The writer determined first by his own specifications that the arch structure would be cheaper than a truss bridge; but, afterward, when it was detailed according to the rulings of the government specifications, the economics were reversed; nevertheless, for several good and sufficient reasons, the arch type of construction was finally adopted. Notwithstanding this fact, the writer cannot agree with Mr. Molitor in his statement that "all that can reasonably be required is to design each member so that it will withstand safely its maximum and minimum conditions of stress, one at a time, without exceeding the allowable units of stress for each condition." Such complete ignoring of stress reversion does not appear to the writer to be warranted by the present professional knowledge concerning the wearing effects of reversing stresses.

Why Mr. Molitor pins his faith to Professor Howe's ancient findings concerning the comparative economics of hingeless, two-hinged, and three-hinged arches, notwithstanding the fact that the writer has lately found, from actually computed designs and weight estimates, diametrically opposite results to those of Professor Howe, is not apparent.

No, the writer certainly is not swinging over to an approval of the use of redundant members in bridge designing; nor does anything in the paper warrant the drawing of such a conclusion.

Replying to Mr. Stoney's statement that the arch bridge is not suitable for India, because the bridges in that country are generally over sand and silt river beds, requiring very deep foundations, the writer would reply that, for such crossings, the arch is absolutely unsuitable; but, when the mountain districts of that country are developed by railroads and highways, the arch type of structure may be found to be appropriate for many proposed crossings of streams and gulches.

Supplementing Mr. Frankland's remarks on the economics of the cantilever arch, the writer would point out that the reduction of the horizontal thrusts at the tops of the piers effects a decided saving in the cost of the latter; and that the greater the proportionate lengths of the flanking spans to the arch spans, the greater the economy in pier material. As before indicated, it is impracticable, in a general study of bridge economics, to take into account the variation in costs of substructures. One could do it by passing from the general to the particular, assuming a foundation profile; but the results deduced would apply numerically to only the crossing assumed. However, they

would give a fair indication of what one might properly expect from other crossing conditions not too dissimilar. Mr. Waddell.

In the case cited by Mr. Frankland, concerning the foundations of an arch bridge being partly rock and partly piles driven to rock, his solution of the problem is perfectly proper, because the bascule separates the two types of foundation; but, as he states, the placing of one abutment of an arch span on rock and the other on piles is an unwise expedient. In the writer's opinion, so doing might, under certain conditions, involve a violation of good engineering practice.

Some engineers, who adhere to old-fashioned notions, contend that masonry or concrete arch bridges are applicable to bed-rock foundations only, and that the piers and abutments therefor should never rest on piles. Although it is true that bed-rock is the ideal foundation for such structures, pile foundations can be made entirely satisfactory, provided the masonry of the bases is carried well down into firm material capable of resisting properly the horizontal thrust. It is not legitimate to count on the horizontal resistance of the piles; and it would be criminal practice to rest the piers and abutments of an arch bridge on piles the tops of which, below the pier bases, act like stilts because of passing through soft material before reaching a hard bearing.

The writer was once the sole cause of the prevention of the building of an exceedingly long, reinforced concrete, combined railway and highway arch bridge over an arm of the sea where a great and disastrous wash-out had taken place, and in the design for which the bases of the piers and abutments were carried a very short distance below extreme low-water mark, and were supported by piles passing through 10 or 12 ft. of extremely soft material before reaching a layer of earth that had any claim to firmness. How much horizontal resistance could such stilts have offered to thrust produced by the passing of heavy locomotives? Had the structure been built on the proposed layout, an appalling accident, probably involving the loss of many human lives, would certainly have resulted; and it was only by the exertion of the writer's utmost powers of eloquence that the authorities were persuaded to reject the design. In fact, something more effective than eloquence was used; for the ignoring of a prediction of dire disaster by an acknowledged authority on bridge design would have rendered the officials concerned criminally liable in case of the occurrence of the predicted failure—and they were informed of that fact. As the writer was not retained on the work, and knew that he was not going to be, his motives were entirely altruistic; and he now has the satisfaction of feeling that the vigorous stand which he took saved the railroad companies and the community from serious trouble and the Engineering Profession in America from a lasting disgrace.

Mr. Waddell.

Mr. Frankland is quite right when he states, in respect to arch designing, that "the proper determination of the true economics calls for more judgment on the part of the designing engineer than with any other type of bridge"; and the writer trusts that the result of this paper, with the discussions and the résumé thereof, will be to enable those engineers who are considering the adoption of the arch type of construction for any crossing to come quickly to correct conclusions concerning the economics involved.

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Paper No. 1427

VERIFICATION OF THE BAZIN WEIR FORMULA BY HYDRO-CHEMICAL GAUGINGS*

By Floyd A. Nagler, Jun. Am. Soc. C. E.†

With Discussion by Messrs. Clemens Herschel, David A. Molitor, Frederic P. Stearns, B. F. Groat, and Floyd A. Nagler.

Synopsis.

This paper presents the results of 23 experiments on a standard Bazin weir, 3.72 ft. high and 6.56 ft. wide. The experiments were carried on in a flume built for the University of Michigan at the Argo Spillway, Ann Arbor, Mich.

The water was measured hydro-chemically, according to the methods recommended by B. F. Groat, M. Am. Soc. C. E., in his paper entitled "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators."‡

Tests were made on heads as high as 4 ft. on the weir, thus extending the application of the weir formulas more than 100 per cent.

This paper describes briefly the method of hydro-chemical and weir gauging used in the experiments.

Introduction.

Volumetric experiments on the flow of water over sharp-crested weirs have been performed for only a comparatively small range in head on the weir. M. Bazin's experiments cover a range in head from 0.194 to 1.780 ft.; those of the late J. B. Francis, Past-President, Am.

* Presented at the meeting of March 6th, 1918.

† Now Assoc. M. Am. Soc. C. E.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 951.

Soc. C. E., are included between 0.736 and 1.060 ft.; and the experiments of Messrs. Fteley and Stearns are between 0.0735 and 1.6038 ft.

These experimenters were unable to use higher heads due to the fact that their volumetric measuring basins filled too rapidly with the larger quantities of water which flowed over the weir at high heads. Nevertheless, these few experiments, within so small a range in head, were used by each experimenter in evaluating the coefficients for the standard weir formulas which bear their names.

For lack of a better basis of computation, engineers have been continually forced to apply the Bazin, Francis, and Fteley and Stearns formulas in the solution of problems involving heads on the weir many times that of the maximum on which any of these formulas are based.

Considerable might be written on the discrepancies (of as much as 3%) which exist between the three formulas for some heads of less than 2 ft.; but nothing is known of the possible differences which may exist for higher heads. Without further experiments, covering a wider range in head, these formulas cannot be applied with security in the computation of discharge in case the head falls outside those limits on which each respective formula is based.

Within the past decade the hydro-chemical method of gauging large quantities of water, originally used by some Swiss engineers, has been introduced into the United States. In 1914 Mr. Groat applied this method with considerable accuracy in testing hydro-electric generators, where quantities of water as large as 1 500 cu. ft. per sec. had to be measured. He published the results of these tests, together with an exhaustive treatment of the theory of salt-solution gauging, in his paper, "Chemi-Hydrometry and Its Application to the Precise Testing of Hydro-Electric Generators". In this paper Mr. Groat describes a method of "salt-solution gauging" for which he claims an accuracy that could only be attained by the most precise volumetric measurements. This method might easily be applied in gauging the flow of water over a weir, thus checking the existing volumetric experiments and extending the present experimental data to higher heads.

Contemplating the performance of such a set of experiments, an experimental flume was constructed at the Argo Spillway, Ann Arbor, Mich., for the University of Michigan, by the Detroit Edison Company. In the construction of this flume, it was decided to duplicate the conditions of the Bazin experiments, with additional capacity

sufficient to obtain heads on the weir (3.72 ft. high and 6.56 ft. long) as high as 4 ft. The proposed experiments could then be compared with an accepted standard weir formula and its basic experiments.

The experiments were performed between March 24th and May 2d, 1917.

Bazin's Experiments and Formula.

Bazin began his extensive weir experiments in 1886 on the Canal de Bourgogne, near Dijon, France. The weirs were constructed in a channel 6.56 ft. (2 m.) wide. Contractions were suppressed. The crest itself was made of an iron plate. Of these experiments 67 were performed on a weir 3.72 ft. high and 6.56 ft. wide, including heads up to 1.012 ft.; 38 on a weir 3.27 ft. high and 3.28 ft. long, with heads up to 1.34 ft.; and 48 on a weir 3.297 ft. high and 1.64 ft. long, including heads up to 1.780 ft. Having calibrated his standard weir within at least 1% by volumetric measurements, M. Bazin proceeded to perform experiments on weirs of different heights at places down stream in the same channel. From the results of these observations, he evaluated the coefficients for his formula. If H is the measured head on the weir, p the height of the weir, and L the length of the weir crest, then Q, the discharge in cubic feet per second, is expressed as follows:

$$Q = \left(0.405 + \frac{0.00984}{H}\right)\left(1 + 0.55\frac{H^2}{(p + H)}\right) L H\sqrt{2gH}$$

If the salt-solution method of gauging in the proposed experiments checks the volumetric measurements of flow over the weir throughout the range of the Bazin experiments, there is then established a basis for measuring accurately the water flowing over the weir, and the range of application of the weir formula becomes extended.

Chemi-Hydrometry.

The process of measuring the quantity of a liquid by observing the dilution that a solution of some chemical undergoes in being thoroughly mixed with that liquid, has been called "chemi-hydrometry."

For example, let 10 lb. of salt be thoroughly dissolved in a tank of water. If, on chemical analysis, it is found that each cubic foot of this water contains 1 lb. of salt, the volume of water in the tank is equal to the ratio of dilution of the salt, or 10 cu. ft. Or, suppose that a salt solution is introduced into a flowing stream at the rate of 1 cu. ft. per sec. If this salt solution contains 10 lb. of salt per cubic foot, and

it is found that each cubic foot of the flowing water, after being mixed with the salt solution, contains 1 lb. of salt, then the rate of flow of the stream is represented by the "ratio of dilution" of the salt solution, multiplied by the rate at which that solution is introduced into the stream; or, in this case, 10 cu. ft. per sec.

These examples give the essence of the theory of hydro-chemical gauging. In practice, however, other factors enter, which produce many complications. For a full treatment of errors and complications which arise, reference should be made to the paper by Mr. Groat previously mentioned. The following brief treatise on the subject is offered in order to save the reader the labor of reading and mastering that voluminous paper, since only that part of the theory is presented which will be necessary for an understanding of the methods used in these experiments. The symbols and definitions used by Mr. Groat have been adopted throughout.

In most cases the normal water contains an initial quantity of salt in solution; this must be taken into consideration, in order to make a correct gauging. Corrections must also be applied in comparing solutions of unlike temperatures. In some cases, cognizance must be taken of the fact that, when two solutions with different percentages of salt in solution are mixed, the total volume is less than the sum of the volumes of the two initial constituent solutions. This shrinkage in volume is taken into consideration by applying a coefficient, k, to the stronger solution, such that the volume of the weaker solution plus the volume of the stronger solution multiplied by k is equal to the volume of the resultant mixture.

The method of "special dilutions" and "balanced evaporations" was adopted in making the salt analysis in these tests. This method simply implies that a special dilution of the salt solution sample with the normal water sample is made in the laboratory. Effort is made to have the "ratio of dilution" of this special dilution approximate the estimated ratio of dilution during the test. This special dilution receives the same method of treatment in the laboratory as the dosed water samples.

Definitions and Equations.—The "ratio of mixture" is the ratio of the quantity of the weaker of two solutions to the quantity of the other solution with which it is mixed. It is represented by r.

The "ratio of dilution" is the ratio of the quantity resulting from the mixture of two salt solutions of different percentages to the quantity having the higher percentage. It is represented by R.

When a solution is referred to as "being of a certain percentage" it is meant that this percentage of the total weight of the solution is dry salt. It is represented by p.

By "concentration of solution" is meant the weight of chemical in solution per unit of volume. It is represented by c.

Then, let Q_1 = the quantity of water flowing in the stream;
q = the rate of discharge of the salt solution;
Q_2 = the quantity of water flowing in the stream after the introduction of the salt solution;
k = the coefficient of shrinkage of the salt solution; and
c_1, c, c_2 = the concentrations of the normal water, salt solution, and dosed flume water, respectively.

If all the solutions are at the same temperature; or, if originally at different temperatures, temperature corrections have been previously applied, then

Q_1c_1 = the weight of salt in the normal water; qc = the weight of salt in the salt solution; and Q_2c_2 = the weight of salt in the mixture.

Hence, $$Q_1c_1 + q\,c = Q_2c_2 \quad \ldots\ldots (1)$$

Also, by definition, $$Q_1 + k\,q = Q_2 \quad \ldots\ldots (2)$$

Likewise, $$\frac{Q_1}{q} = r$$

and, $$\frac{Q_2}{q} = R$$

By substitution in Equation (1)

$$rc_1 + c = Rc_2 \quad \ldots\ldots (3)$$

By substitution in Equation (2)

$$r + k = R \quad \ldots\ldots (4)$$

By substituting Equation (4) in Equation (3), we obtain,

$$rc_1 + c = rc_2 + kc_2$$

Whence, solving for r, we obtain,

$$r = \frac{c - kc_2}{c_2 - c_1} \quad \ldots\ldots (5)$$

Also, solving for c_1, we obtain,

$$c_1 = \frac{rc_2 + kc_2 - c}{r} \quad \ldots\ldots (6)$$

Equation (5) may be applied when the method of special dilutions is not used. It is applicable when the normal water, dosed water, and salt-solution sample are analyzed directly.

However, let a special dilution be made up in the laboratory, using the actual salt-solution sample with the concentration, c, and the normal pond water with the concentration, c_1. The ratios of mixture and dilution should approximate r and R, respectively, but actually have values of r' and R'. The final dilution has a concentration c'_2, approximating c_2. Then, by analogy, we may write, from Equation (6),

$$c_1 = \frac{r'c'_2 + kc'_2 - c}{r'}$$

In this equation, r must approximate r' so closely that the coefficient of shrinkage is the same for both dilutions.

Equations (6) and (7), however, may be placed equal to each other.

$$\frac{r'c'_2 + kc'_2 - c}{r'} = \frac{rc_2 + kc_2 - c}{r}$$

Solving for r, we obtain

$$r = \frac{r'}{\dfrac{c - r'c_2 - r'c'_2 - kc'_2}{c - kc_2}}$$

By analogy, from Equation (4), however, $r' = R' - k$. Substituting this value of r' in the denominator of the previous expression, we obtain

$$r = \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - kc_2}}$$

Thus,
$$Q_1 = qr = q \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - kc_2}} \qquad (8)$$

This equation is of practical application. It should be observed that c_1 has been eliminated; hence, it is unnecessary to analyze the normal pond water for its saline content. It has been stated that the use of this equation requires the previous correction of the volume of every solution handled throughout the entire laboratory process, reducing every sample analyzed in each experiment to some common temperature.

The reagent used in salt analysis is silver nitrate. This chemical is dissolved in distilled water in some standard proportions. Then a

measured quantity of this silver nitrate solution is used, sufficient to neutralize exactly all the salt in a given sample. The molecular weight of silver nitrate is 170; that of sodium chloride (common salt) is 58.5. Hence, 1 gramme of silver nitrate will neutralize $\frac{58.5}{170} = 0.3441$ grammes of salt. Thus, if the quantity of anhydrous silver nitrate contained in the silver nitrate solution used in a titration is known, the quantity of anhydrous salt contained in the sample is 0.3441 times this quantity.

If all titrations require about the same quantity of silver nitrate solution, and all samples, prepared for analysis, are of the same character and volume, and if the same quantity of indicator is used to indicate the end point of the reaction for each sample, then, for practical purposes, we may write,

$$c = \alpha\, t,$$

where t is the volume of the silver nitrate solution required to titrate a unit volume of the sample, and α is a constant dependent on the concentration of the standard silver nitrate solution, and the actual equivalent between the silver nitrate and the sodium chloride. Hence, we may write with practically (not theoretically) no error,

$$Q_1 = q\, r = q\, \frac{r'}{1 + R' \frac{t_2 - t'_2}{t - k\, t_2}} \qquad (9)$$

Thus, if silver nitrate of the same strength is used for all titrations in a single experiment, it becomes unnecessary to determine the actual salt content of the sample or the exact concentration of the silver nitrate solution. The values of t'_2 and t_2 have such a relation in the equation that when t'_2 approximates t_2, and there is then the same relative error in both titrations, this error has no effect on the practical value of r.

Therefore, in order to make a salt-solution test, three samples must be provided:

(1) A sample of the salt solution;

(2) A sample of the normal pond water; and,

(3) A sample of the dosed flume water.

The equation also designates that q, the rate of dosing, must be observed; and the laboratory process further involves the observation

of the temperatures of all solutions and the calibration of all flasks and pipettes, in order that r' and R' may be calculated with accuracy.

Description of Testing Plant.

Testing Flume.—The flume in which these experiments were performed was built in September, 1916, by the Detroit Edison Company. Connecting with two of the flood-gates at the Argo Spillway, it extended 143 ft. down stream.

Figs. 1 to 4 show the nature of the design and the construction of the flume.

The first 40 ft. of the flume was built 8½ ft. wide and 9 ft. deep. It then became necessary to divert it in the direction assumed by the natural flow of the river, and the remaining 103 ft. was made 6.56 ft. (2 m.) wide, the width of the Bazin channel. The lining was of 2-in., matched yellow pine, with driven splines uniting every board. No effort was made to caulk these joints or fill them with white lead. After the flume had been filled with water for three days, allowing the lining to soak and expand, there was no visible leakage at any point.

Figs. 5, 6, and 7 show the flume in process of construction, and Fig. 9 is reproduced from a photograph of the finished structure.

The quantity of water entering the flume was regulated by lowering or raising the flood-gates at the spillway. The salt solution was introduced under pressure into a sprinkling system in the orifices of these gates. Due to the fact that the water entered the flume at a very high velocity, it first became necessary to lower this abnormal velocity and distribute it uniformly over the entire flume section. This was accomplished by constructing a submerged weir 3½ ft. high across the flume at a point 16 ft. down stream from the gates. In striking this obstruction at a high velocity, the water was thrown into a great commotion, passing over the weir with a fair velocity distribution, regardless of the turbulence. This weir proved an excellent mixer for the salt solution, securing practically its uniform distribution throughout the entire flume section.

As the water passed around the bend in the flume, the velocity at the outside was observed to be perceptibly higher than that on the inside. This abnormal condition was remedied by placing sets of horizontal and vertical baffles at a point 10 ft. below the bend. The

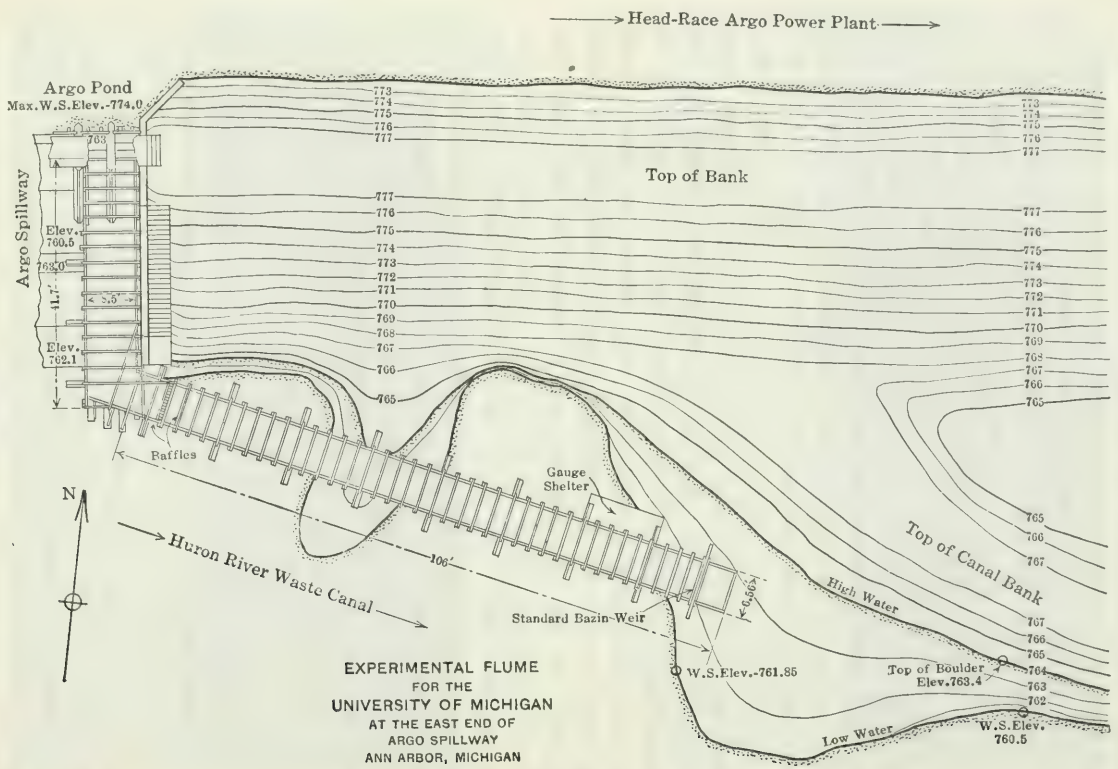
Head-Race Argo Power Plant
Argo Pond
Max. W.S.Elev.-774.0
Argo Spillway
Elev. 760.5
41.7'
Elev. 762.1
Top of Bank
Baffles
Gauge Shelter
N
Huron River Waste Canal
106'
Standard Bazin Weir
6.66'
Top of Canal Bank
High Water
W.S.Elev.-761.85
Top of Boulder Elev.763.4
Low Water
W.S.Elev. 760.5
EXPERIMENTAL FLUME
FOR THE
UNIVERSITY OF MICHIGAN
AT THE EAST END OF
ARGO SPILLWAY
ANN ARBOR, MICHIGAN

FIG. 1.

baffles were constructed of 2-in. yellow pine boards, swinging on an eccentric longitudinal axis. By closing the vertical baffles at the right of the flume, a uniform distribution of velocities was again secured. Fig. 11 exhibits velocity contours taken at a point 25 ft. above the weir. These contours were obtained with an Ott current meter operating on a metal rod when there was a head of about 3 ft. on the weir.

Two floating railroad ties were placed across the flume about 28 ft. above the weir. These were efficient in reducing the small oscillatory wave motion of the water in the channel of approach.

The Weir.—The weir was built up of 4-in. timbers, as shown in the section on Fig. 4. The crest was formed by a $\frac{1}{8}$-in. brass strip, inserted at one edge of, and flush with one face of, a 7 by $\frac{3}{8}$-in. plate of cold-rolled steel. The brass strip was soldered and screwed to the steel plate, all recesses in the countersunk holes being filled with solder. The back face of the plate and the upper edge of the brass was then planed and finished absolutely true. This left a sharp right-angled knife-edge on the up-stream corner of the crest. The object of the brass strip was to produce a crest which would not be subject to corrosion, thus preventing the introduction of error in the weir discharge from this source. The whole plate was inserted flush with the up-stream face of the wooden timbers, and fastened to them by bolts the heads of which were countersunk in the back of the steel plate. The crest was set at an elevation of 3.72 ft. above the bottom of the flume.

Four $\frac{3}{4}$-in. bolts, 40 in. long, bound the weir timbers to the bottom of the flume. By loosening or tightening the nuts on these bolts, on either side, the weir crest was maintained absolutely level throughout the experiments.

The sides of the flume extended 6 ft. beyond the weir, but the floor extended no farther than the weir. This feature insured abundant aeration for the nappe from underneath.

The frame of the flume at the weir section was made of quadruple strength. All joints were bolted, and great care was exercised in plumbing the side posts. The timbers under the flume, at this point, rested on 4 ft. of concrete. The side posts of the frame extended for 3 ft. into the concrete. This produced a very rigid and permanent frame at the weir section. The construction is clearly shown in Fig. 4, and the erection of the frame is shown in Fig. 6.

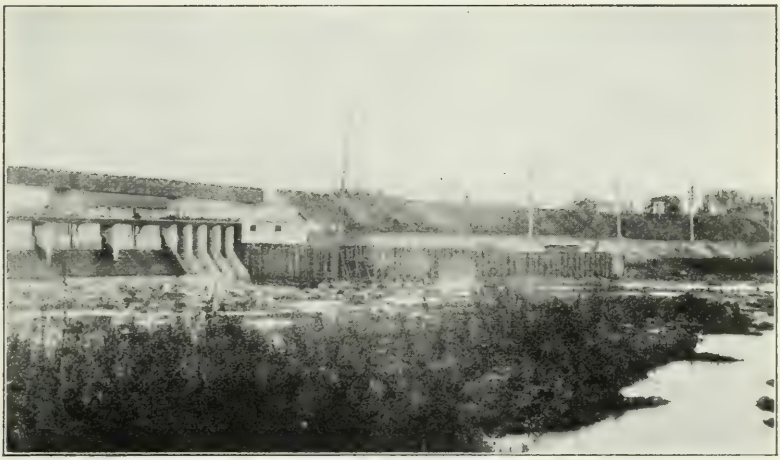

FIG. 2.—UNIVERSITY OF MICHIGAN EXPERIMENTAL FLUME.

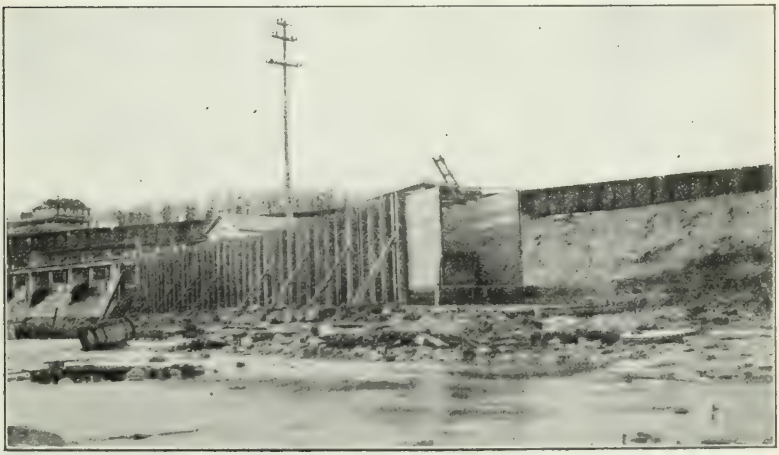

FIG. 3.—UNIVERSITY OF MICHIGAN EXPERIMENTAL FLUME.

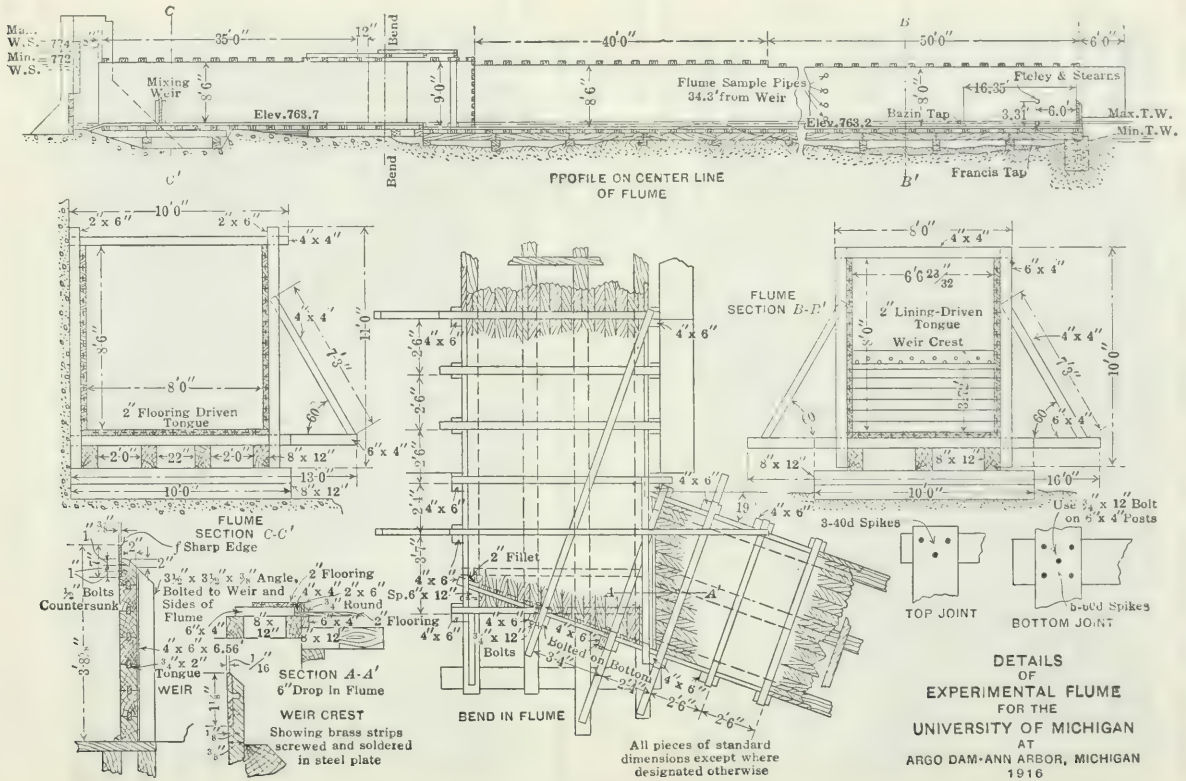
PROFILE ON CENTER LINE OF FLUME
Mixing Weir
Elev.763.7
Flume Sample Pipes 34.3' from Weir
Elev.763.2
Bazin Tap
Fteley & Stearns
Francis Tap
Max.T.W.
Min.T.W.
FLUME SECTION C-C'
Sharp Edge
2" Flooring Driven Tongue
SECTION A-A'
6" Drop in Flume
WEIR CREST
Showing brass strips screwed and soldered in steel plate
BEND IN FLUME
FLUME SECTION B-B'
2" Lining-Driven Tongue Weir Crest
3-40d Spikes
TOP JOINT
BOTTOM JOINT
Use ¾" x 12" Bolt on 6" x 4" Posts
All pieces of standard dimensions except where designated otherwise
DETAILS OF EXPERIMENTAL FLUME FOR THE UNIVERSITY OF MICHIGAN AT ARGO DAM-ANN ARBOR, MICHIGAN 1916

FIG. 4.

Placing the weir at the end of the flume gave it a free basin, about 90 ft. in length. The bottom of the flume was level throughout this entire length.

Fig. 10 shows the weir with about 0.3 ft. of water flowing over the crest.

Hook-Gauges and Piezometer.—For all heads up to 2 ft., the head on the weir was measured simultaneously by three hook-gauges operating in galvanized tanks connected to the flume by 2-in. pipes. These pipes were attached so that the opening was flush with the inside wall of the flume. Heads of from 2 to 4 ft. were measured by a fourth hook-gauge. A piezometer tube, with a $\frac{1}{4}$-in. opening, flush with the side of the flume, was also read as a check on the hook-gauge measurements.

Hook-gauge No. 1 was one of the new all-metal Gurley gauges, with a 45° point. The stilling tank for this gauge was connected to a point 6 ft. from the weir and 0.414 ft. below the weir crest. This was the point of observation used by Fteley and Stearns.

Hook-gauge No. 2 was one of the old Gurley gauges, with wooden slides and a sharp needle point. The stilling tank for this gauge was connected to a point 6 ft. from the weir and 6 in. above the bottom of the flume. This was the point of observation used by Francis.

Hook-gauge No. 3 was also one of the old Gurley gauges, with wooden slides, but it was equipped with a 45° point. The stilling tank for this gauge connected to points on both sides of the flume, 16.35 ft. from the weir and 6 in. above the floor of the flume. This was the point of observation used by Bazin.

Hook-gauge No. 4 was an all-metal hook-gauge designed especially for these experiments, and made by Eberbach and Son, of Ann Arbor, Mich.; it was found to be more satisfactory in its operation than any of the four gauges used. The stilling tank for this gauge connected with the lead pipes to Gauge No. 3.

The piezometer tube was $1\frac{1}{2}$ in. in diameter, with a brass scale, graduated to 0.005 ft., mounted at its side. The tube was connected with the flume at a point 11 ft. from the weir and 6 in. below the crest.

Each gauge was provided with an electric light on a drop cord in order to assist in the observation of the reflection at the point. Gauges with the 45° point were found to be the most readily observed.

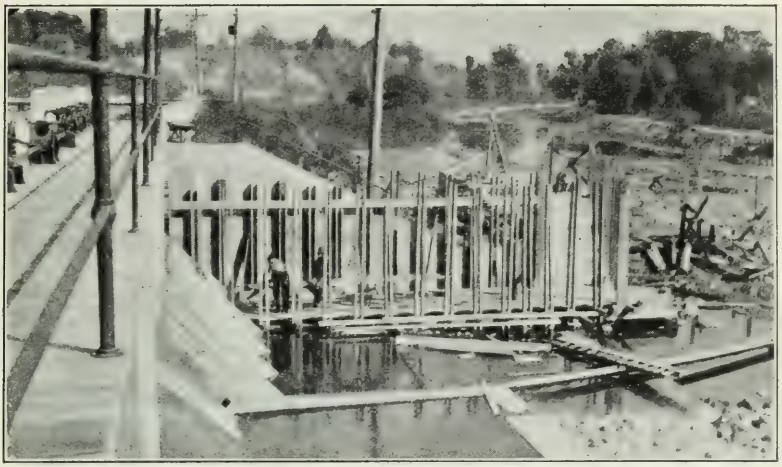

FIG. 5.—CONSTRUCTION OF FLUME.

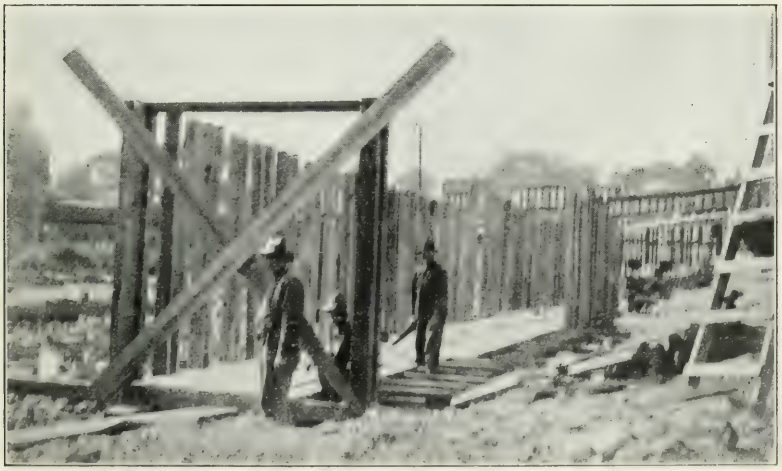

FIG. 6.—CONSTRUCTION OF FLUME.

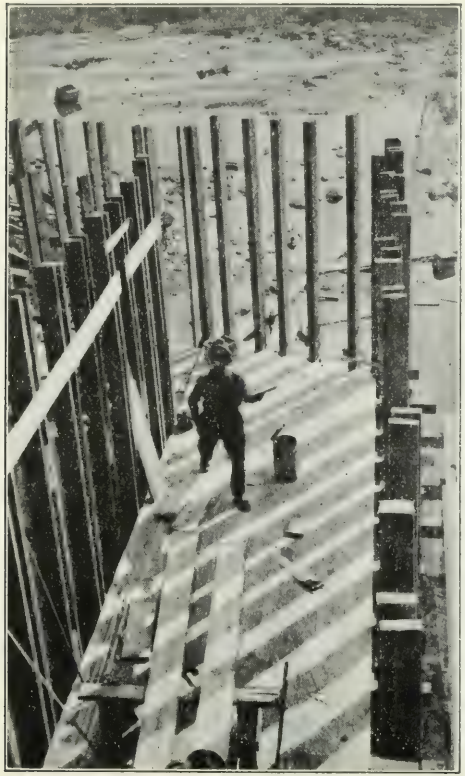

FIG. 7.—CONSTRUCTION OF FLUME.

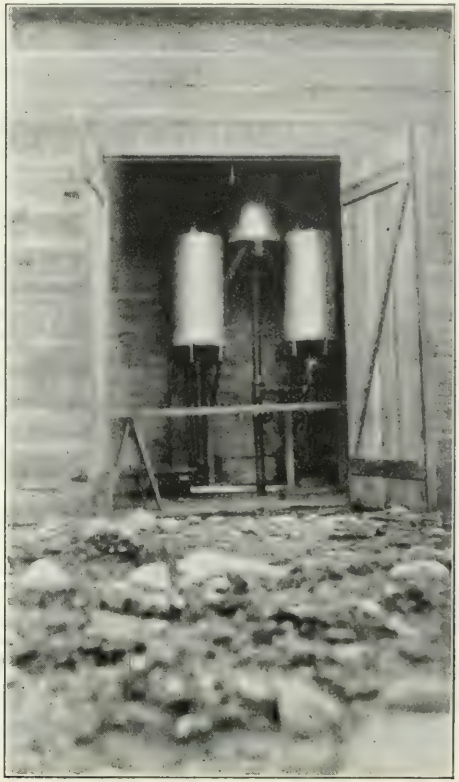

FIG. 8.—WEIR GAUGE SHELTER.

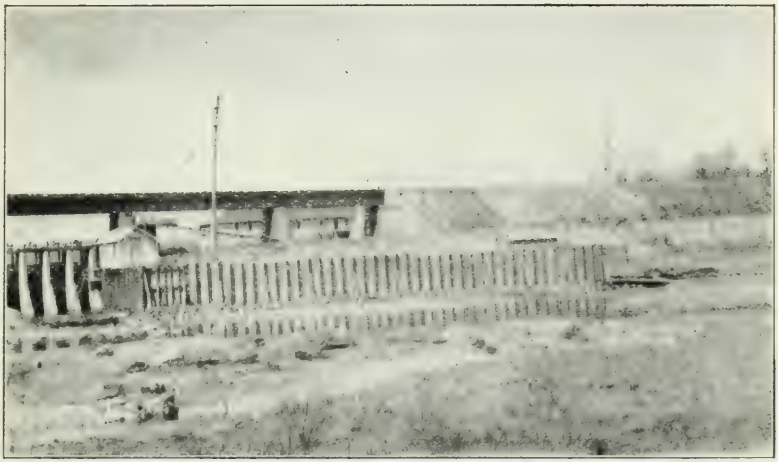

FIG. 9.—THE ARGO TESTING FLUME.

FIG. 10.—THE WEIR.

All hook-gauges were mounted on a frame of 4 by 4-in. timbers. This frame was independent of the flume, having its foundation in a mass of concrete.

The gauges were enclosed within a gauge shelter for protection. Fig. 8 shows a little of the interior of this shelter, three of the galvanized stilling tanks being plainly visible.

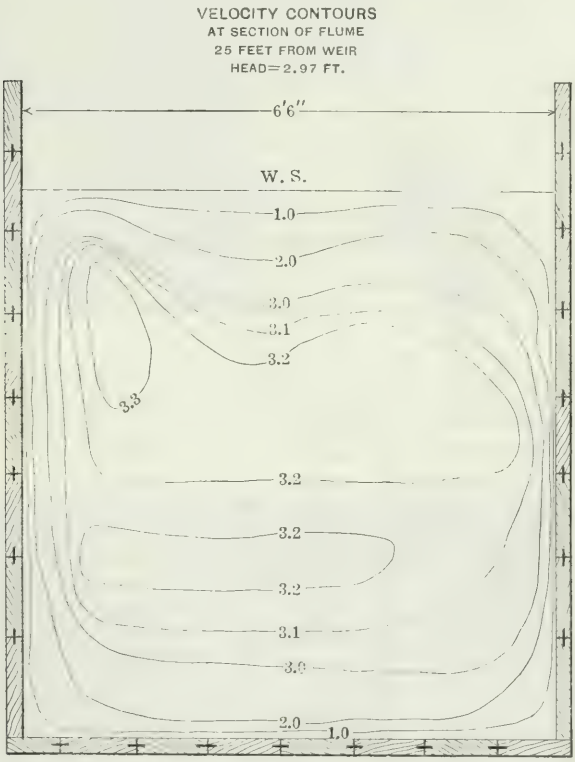

FIG. 11.

Dosing Apparatus.—In order to be able to measure a discharge over the weir as great as 200 cu. ft. per sec., by diluting a salt solution of a concentration of 300 grammes per liter, in the ratio of 6 000 to 1, during an experiment lasting 15 min., it was predetermined that a salt-solution tank with a capacity of about 50 cu. ft. would be required. This allowed 5 min. for dosing before and after each experiment, in order to insure good regulation of the dosing rate.

The solution tank is shown by Fig. 12. The tank was 4.067 by 3.33 ft. and 4 ft. deep. A piezometer was mounted on its side. The glass tube was 1 in. in diameter, and connected with the bottom of the tank. The height of the column of solution standing in the tube indicated the elevation of the solution in the tank. The gauge scale was made by Eberbach and Son. The scale had a sliding vernier

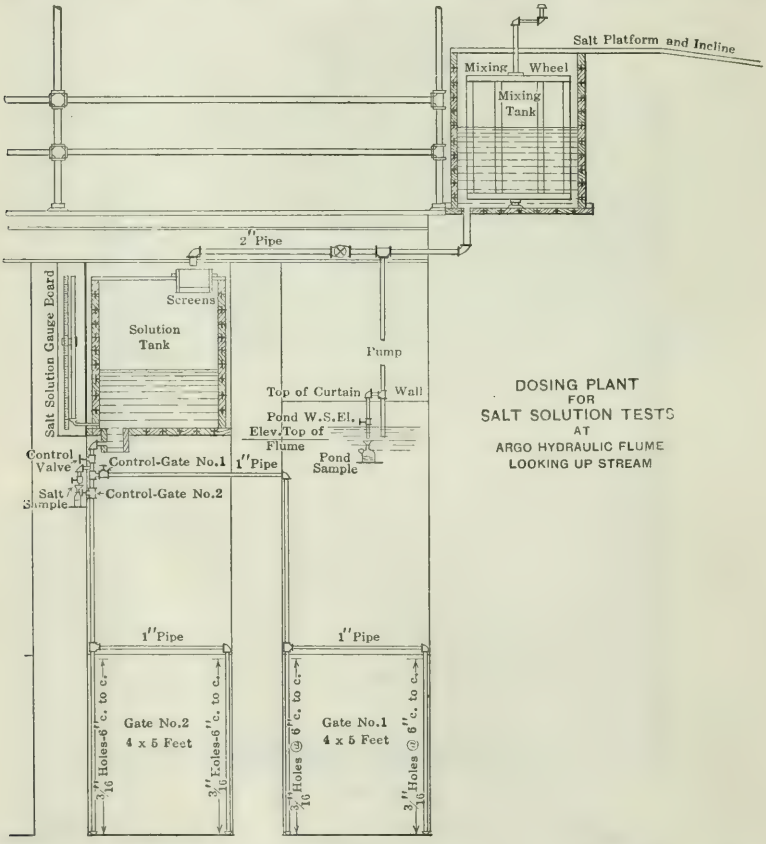

FIG. 12.

attached to a mirror index. It was possible to read the water-surface elevation within 0.001 ft. with this apparatus. Fig. 13 shows the location of the solution tank and gauge.

In varying the discharge of the water through the flume, from the least to the largest quantities, it was decided to use always the maximum quantity of salt solution which could be obtained from the

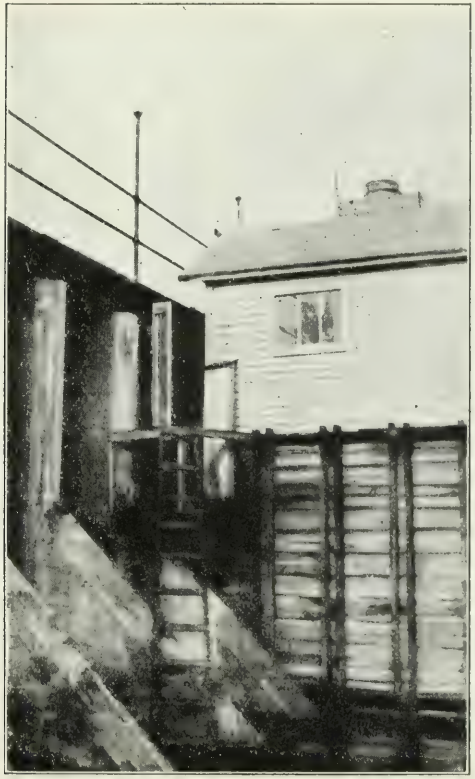

FIG. 13.—LOOKING TOWARD THE SOLUTION TANK AND PIEZOMETER.

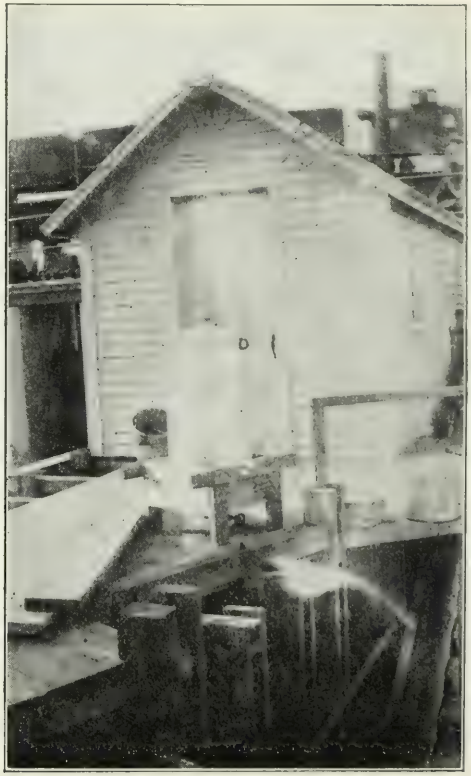

FIG. 14.—THE CALIBRATOR.

tank. This would insure a maximum difference between the initial and final readings of the piezometer, reducing to a minimum the effect of an error in the absolute reading of the piezometer scale. Hence, an endeavor was made to maintain q approximately the same for all experiments. Consequently, in order that the flume samples from all experiments might have the same concentration, it was necessary to vary the concentration of the salt solution for each experiment. The quantity of salt used varied from 1 000 lb. for the highest, to 15 lb. for the lowest, head.

Ordinary commercial salt was used. It was found to contain some foreign matter, including small chips and fine sand. This fact, combined with the requirement that no undissolved salt be present in the solution tank, necessitated first dissolving the salt in a mixing tank and then screening off all foreign matter before passing it into the solution tank. Screens of 30 and 80 meshes to the inch were used.

The mixing tank was filled with water by the pump indicated on Fig. 12. The solution of 1 000 lb. of salt was accomplished in only 40 min. with the aid of the mixing wheel inside the mixing tank.

The rate of dosing the salt solution was maintained constant throughout the entire experiment by a general control valve. Beyond this point, provision was made for drawing off a continuous sample of the salt solution through a small pipe tapping the larger discharge pipe. The salt solution could be introduced into Gate No. 1 or Gate No. 2 in any quantity desired, by the regulation of the valves in the pipes conducting it to them.

The salt solution was introduced into the orifice under the pressure head available due to the elevation of the solution tank. Distributing pipes, 1 in. in diameter, were fastened on each side of the orifice, and $\frac{3}{16}$-in. holes, 6 in. from center to center, were drilled in these pipes at an angle of about 30° down stream. These distributing pipes, combined with the submerged weir and baffles, produced a very uniform distribution of the salt solution throughout the entire flume section.

Fig. 15 shows the general plan of the entire testing plant. The mixing tank and salt platform are in the foreground.

Sampling Apparatus.—The dosed flume-water samples were taken at a section 34.3 ft. above the weir, or 108.7 ft. from the point at which the salt solution was charged into the water.

With the flume sampling apparatus, Fig. 17, two to four samples of dosed flume water were obtained from 8 to 16 points in the flume section. Each of the inclined pipes had four $\frac{3}{64}$-in. holes drilled in the upper side, 1.64 ft. apart. The discharge through each of these small orifices was "free" and practically the same throughout the length of the pipe. The water then flowed down the inclined pipe into the sample bottle at the left. The valve at the right provided means for flushing the pipe, the pet-cock above it remaining open when the pipe was not being flushed. An excess of the sample, above the predetermined rate of 1 gal. in 15 min., for which the pet-cocks at the left were set, flowed out of the pipe through the by-pass above the pet-cock. Under any head, this excess did not exceed the sampling rate.

A sample of the normal pond water was drawn from the suction pipe of the pump, as shown by Fig. 12.

Electric Bell and Salt Detection Apparatus.—There was a 5-volt battery circuit with electric bells at the solution tank and in the gauge shelter. These bells could be heard from any point on the flume. The ringing of the bells was controlled by a push-button in the laboratory.

A salt detection circuit was established in parallel with the bell circuit. The contact points, $\frac{1}{8}$ in. apart, were inside the flume, 3.5 ft. from the bottom, at the bend, at the sampling section, and at the Bazin gauging section. A milli-voltmeter in the laboratory was connected in this circuit so that it could be placed in series with any one set of contacts. With normal water flowing through the flume, the instrument registered 15 milli-volts; but when the dosed flume water reached the contacts, the pointer would swing slowly up to 19.5 milli-volts, the salt having increased the conductivity of the water. This instrument proved invaluable in determining the time interval required by the salt to travel from the point of dosing to that of sampling. From the indications of the milli-voltmeter, the "timer" could signal the sampler when the pointer showed that a uniform regime of sale dosing had been established.

Metal Calibrator.—A heavy galvanized-iron calibrator of 4.262 cu. ft. capacity, at 10.2° cent., was used in calibrating the solution tank. The calibrator had a conical bottom and top, reducing to a

Fig. 15.—The Flume and Salt-Solution Apparatus.

Fig. 16. The Evaporation Apparatus

1¼-in. pipe at each end. In principle, it was nothing more or less than a large pipette. Fig. 14 shows the calibrator standing in front of the laboratory, mounted on a wooden frame for convenient handling.

Chemical Laboratory.—A chemical laboratory, 8.5 by 18 ft., with sufficient equipment to make a salt solution analysis, was built on top of the flume near the spillway. Fig. 14 shows the entrance to the laboratory, and Fig 18 is a view of the inside, as seen from the door.

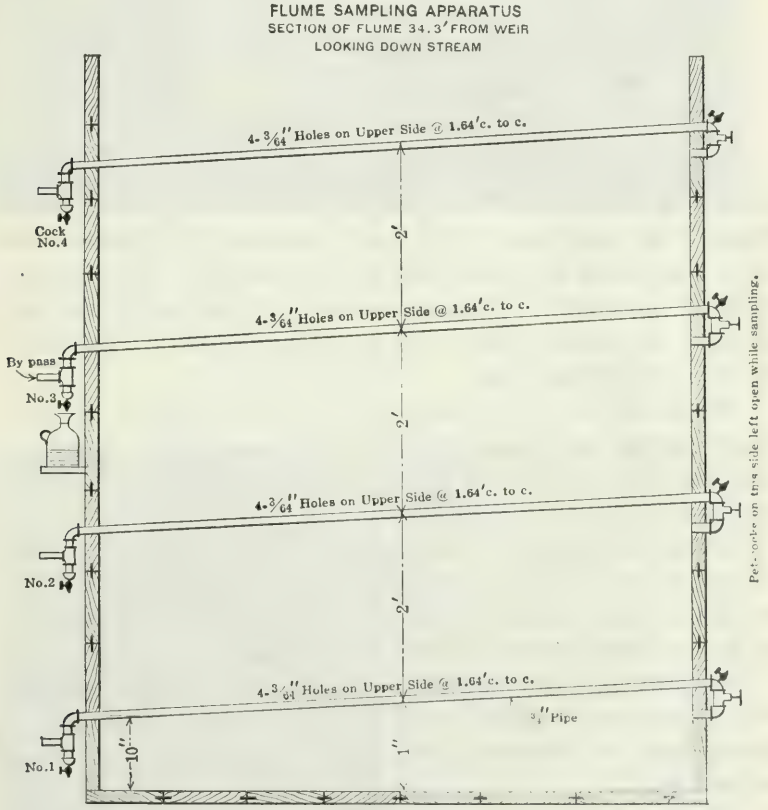

FIG. 17.

The following is a list of the essential apparatus in the laboratory:

1 balance with weights; sensitive to 0.1 mg.
1 Fairbanks scales, of 500 lb. capacity.
2 four-unit electric evaporators.
8 one-half liter separatory funnels.

4 100-cu. cm. burettes at 20° cent.
16 No. 3 casseroles.
3 one-liter volumetric flasks.
2 500-cu. cm. volumetric flasks.
1 100-cu. cm. volumetric flasks.
3 standard thermometers from 0° to 110° cent.
3 10-cu. cm. pipettes.
1 hydrometer.
12 one-liter flasks.
62 one-gallon spring-water bottles.
20 one-quart wide-mouth bottles.
Miscellaneous beakers, rubber stoppers, funnels, etc.

The separatory funnels, casseroles, and evaporators, as mounted during evaporation, are shown in Fig. 16. The evaporators were specially designed and manufactured for these experiments by Eberbach and Son. The heating coils were made of nichrome wire. By connecting a variable resistance in series with each unit, the heat under each casserole was regulated so as to keep the temperature of the sample just below the boiling point.

The experiments were performed so that the quantity of silver nitrate solution used in the titration of all samples would be either a little less or greater than 500 cu. cm. This required the use of 100-cu. cm. burettes, graduated to 0.1 cu. cm.

Spring-water bottles of 1-gal. capacity were used in obtaining the samples of the dosed flume water and the normal pond water. The salt sample was obtained in wide-mouthed 1-qt. bottles. When filled, all sample bottles were corked with rubber stoppers.

The hydrometer proved indispensable in checking the density of the salt solution in order to insure that all salt thrown into the mixing tank was dissolved before drawing the solution off into the solution tank.

The Fairbanks scales were used on top of the salt platform in weighing the salt to be dissolved.

Chemicals.—The chemicals used in the test were commercial salt, silver nitrate, and potassium bichromate.

A good grade of commercial salt was purchased at $1.43 per bbl. In these experiments, 50 bbl. were used.

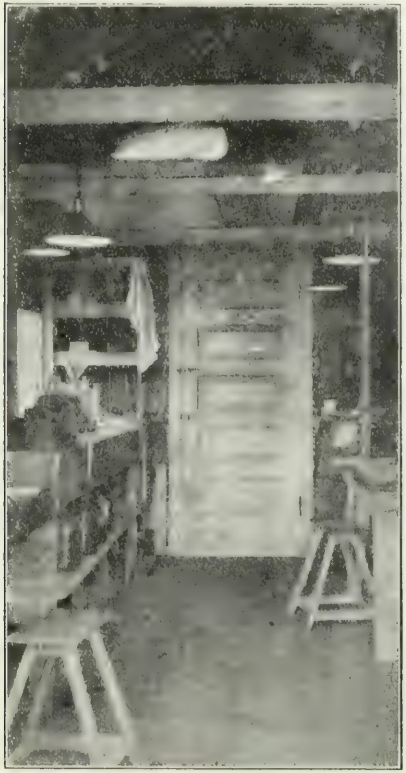

FIG. 18.—INTERIOR OF LABORATORY.

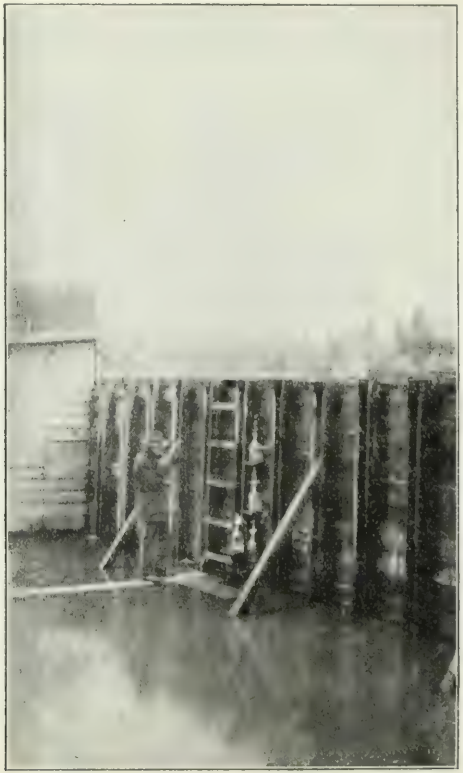

FIG. 19.—TAKING FLUME SAMPLES.

Silver nitrate was used as the reagent to detect the presence of the sodium chloride. A stock solution was made by dissolving 15 grammes of chemically pure silver nitrate in a liter of distilled water. This solution was kept in a dark-colored glass bottle in a dark closet, in order to prevent action by light. This stock solution was diluted in the ratio of 10 to 1 in making the silver nitrate solution used in titration. This solution was also kept away from the light.

A potassium bichromate solution, having a concentration of 50 grammes per liter, was used to indicate the end point in the reaction of the silver nitrate on the sodium chloride. Only 6 or 7 drops of this solution were used for each titration. The indicator gives a lemon yellow color to the sample when the sodium chloride is in excess, turning to a light orange tint the instant the silver nitrate is in excess.

Hydrochloric acid proved indispensable in cleansing the casseroles after evaporation and titration. However, after cleansing, care was taken to wash away the last traces of the acid with distilled water before using the casserole for another evaporation.

Distilled water was obtained from the Chemical Laboratory of the University of Michigan. It was used in making the silver nitrate solutions, potassium bichromate solution, salt solution, sample dilutions, and in washing and cleansing the apparatus.

Method of Experimentation.

In the execution of a single experiment it was generally necessary to obtain data on the following quantities:

Salt Solution Gauging.—

(1) Dosed flume water sample;
(2) Normal pond water sample;
(3) Average salt solution sample;
(4) Average rate of dosing salt solution:
  (*a*) Quantity of solution used,
  (*b*) Duration of experiment;
(5) Temperature of flume water and salt solution.

Weir Gauging.—

(1) Average reading of the Fteley and Stearns gauge;
(2) Average reading of the Francis hook-gauge;
(3) Average reading of the Bazin hook-gauge;
(4) Average reading of the piezometer tube.

Subsequent Chemical Analysis.—

(1) Temperatures and volumes of all solutions and samples handled in the laboratory;

(2) Titration, in cubic centimeters of silver nitrate, of all samples.

In order to obtain this information, it was necessary to obtain observers to act in the following capacities:

(1) Supervising engineer,
(2) Assistant engineer,
(3) Salt solution gauge observer,
(4) Dosing valve operator,
(5) Recorder for dosing operation,
(6) Pond sampler,
(7) Timer,
(8) Flume water sampler,
(9) Observer at piezometer tube,
(10) Observer at Hook-Gauge No. 1,
(11) Observer at Hook-Gauge No. 2,
(12) Observer at Hook-Gauge No. 3,
(13) Chemist.

Duties of the Observers.—(1).—The duties of the supervising engineer were to outline and organize the work for the experiment, and then critically to observe its progress. By the curve sheet, Fig. 20, he knew the probable weir discharge and flume velocity for any given head on the weir. He advised the assistant engineer as to the number of pounds of salt required for the experiment, and also the specific gravity of the resultant salt solution. He advised the timer as to the probable time which it would take the salt to travel from the point of dosing to the point of sampling. He advised the chemist as to the ratio of dilution for the salt solution sample and the special dilutions.

(2).—The duties of the assistant engineer were to execute the experiment. He attended to the raising or lowering of the flood-gates, filling the mixing and solution tanks, and dissolving the salt. He took observations on the temperature of the flume water and salt solution; checked the timer on the duration of the experiment; and gave assistance to any observer in emergency.

(3).—The duties of the salt solution gauge observer were to note the readings of the piezometer tube, and thus maintain a constant rate

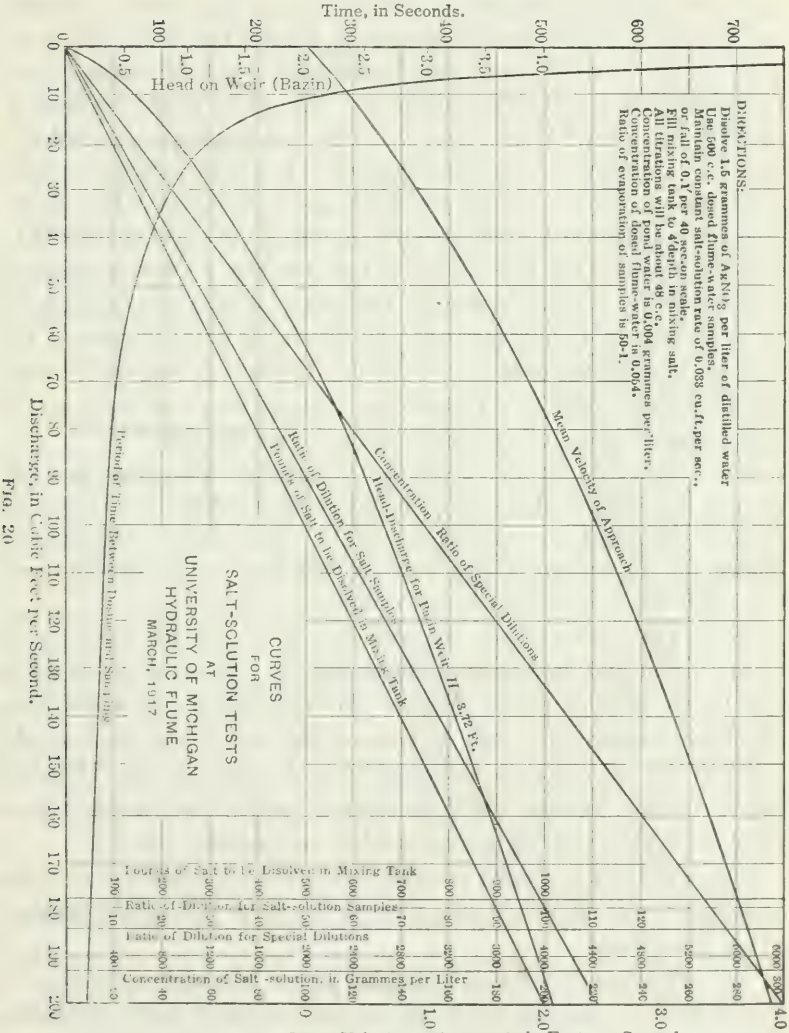
Time, in Seconds.
Head on Weir (Bazin)
DIRECTIONS:
Dissolve 1.5 grammes of AgNO3 per liter of distilled water
Use 500 c.c. dosed flume-water samples.
Maximum flume discharge rate of 0.933 cu.ft.per sec.,
or fall of 0.1' per 40 sec. minimum rate.
Fill mixing tank to 4'depth in mixing salt.
All titrations will be about 48 c.c.
Concentration of pond water is 0.004 grammes per liter.
Concentration of dosed flume-water is 0.0/4.
Ratio of evaporation of samples is 50-1.
Mean Velocity of Approach
Concentration Ratio of Special Dilutions
Head Discharge for Basin Weir II 8.72 Ft.
Ratio of Dilution for Salt Samples
Pounds of Salt to be Dissolved in Mixing Tank
CURVES
FOR
SALT-SOLUTION TESTS
AT
UNIVERSITY OF MICHIGAN
HYDRAULIC FLUME
MARCH, 1917
Period of Time Between Dosing and Sampling
Pounds of Salt to be Dissolved in Mixing Tank
Ratio of Dilution for Salt-solution Samples
Ratio of Dilution for Special Dilutions
Concentration of Salt -solution, in Grammes per Liter
Discharge, in Cubic Feet per Second.
Mean Velocity of Approach, in Feet per Second.

Fig. 20

of dosing throughout the entire experiment. It had been predetermined that the solution in the tank should fall 0.1 ft. in 40 sec. The observer blew a whistle when the dosing was started. He held a stop-watch in each hand; then, advancing the vernier 0.1 ft. at a time, he started one stop-watch at the same time that he stopped the other, calling off to the recorder and the valve man the number of seconds which it took the solution in the tank to fall 0.1 ft. The valve man proceeded to regulate the control valve so as to obtain and maintain the predetermined rate of flow. After good regulation had been secured, he blew a whistle as the column in the tube passed the next 0.1-ft. mark, giving the recorder the scale reading at this point. He continued to give the recorder the time which it took the salt solution to descend each successive 0.1-ft. division on the scale. After the expiration of 15 min., he ended the experiment by blowing a third whistle as the solution in the tube passed the next 0.1-ft. mark, giving the recorder the final gauge reading. The dosing, however, was continued at the same constant rate until the solution tank was emptied.

(*4*).—The duty of the dosing valve operator was either to close or open the control valve so as to maintain the predetermined salt rate. He also secured two 1-qt. samples of the salt solution continuously during the experiment, taking care to flush both the bottles and lead pipe with the salt solution before sampling.

(*5*).—The duties of the recorder were to record the observations of the initial and final scale readings and the time intervals as given him by the solution gauge observer. He also held a stop-watch from which he obtained the total duration of the experiment, starting the watch at the second whistle and stopping at the third whistle. However, the stop-watch was not relied on entirely, as its reading was always checked by observing the second-hand on an ordinary watch.

(*6*).—The pond sampler secured 1 or 2 gal. of normal pond water from the siphon in the suction pipe of the pump during the progress of the experiment.

(*7*).—The timer occupied a position in the laboratory near the bell push-button, so that he could also see the milli-volt meter indicating the presence of salt. He recorded the time of the first whistle, notifying him that dosing had begun. He then observed the time interval before his milli-voltmeter indicated that salt had arrived in any strength at the sampling section. When the salt arrived at the

sampling section in full strength, he rang a long bell. He recorded the time of the second whistle denoting that uniform salt regulation had been attained. Allowing the same period of time after the second whistle as it took the salt to arrive at the sampling section in its prescribed strength after the first whistle, he started the gauge observers reading and the flume sampler sampling by ringing two short bells. Every 20 sec. after this, he rang one short bell, signaling a simultaneous reading of all the weir gauges. He recorded the time of the third whistle indicating that the official dosing had ceased. After a time interval had elapsed, since the third whistle, equal to the time interval which it took for the salt to arrive in any strength at all at the point of sampling after it had left the point of dosing, he finally signaled the flume sampler to cease taking samples and the weir gauge observers to take their final reading. This signal was three long bells. During the entire experiment, he watched the milli-voltmeter, noting any irregularities in its behavior. He checked the difference in time between the second and third whistles with the duration of the experiment obtained by the recorder and the assistant engineer.

(8).—The flume water sampler cleaned out his sampling pipes with compressed air before the experiment was started. This blew all scale and sediment away from the small $\frac{3}{64}$-in. orifices. He then set the sampling pet-cocks so that the quantity discharged through them would fill a 1-gal. bottle in about 15 min. At the ringing of the long bell, he flushed out the sampling pipes and washed the sample bottles with the dosed flume water. He accomplished this as rapidly as possible and placed himself in readiness to start taking samples. He started the samples at the ringing of two short bells, finally withdrawing the bottles when three long bells signaled that the experiment had ceased. He immediately corked the sample bottles and carried them to the laboratory. Fig. 19 shows the sampler watching his samples during the progress of an experiment. The photograph was taken during high water, when the tail-water reached the elevation of the floor of the flume. This condition existed only for one week.

(9), *(10)*, *(11)*, *and (12)*.—All weir gauge observers placed themselves in readiness to read their gauges at the ringing of the single long bell. When two short bells sounded, they took their initial reading, recording the time at which this reading was taken. They then continued to read their gauges every 20 sec., when a single short

bell signaled the reading. They took a final reading when the bell rang three successive times, recording this reading, and also the time at which it was taken.

(*13*).—The chemist had three classes of samples to analyze: the dosed flume water, the special dilutions, and the salt solution sample. The method of balanced evaporations required the dosed flume water and special dilutions to be evaporated and the salt solution samples to be diluted, so that final samples of similar volume of all three classes would require approximately the same quantity of silver nitrate solution in titration.

Flume samples were obtained from as many cocks as were submerged during the experiment. Three analyses were performed on each sample, to insure against error and contamination in the laboratory. As a rule, the three analyses agreed very closely and, in almost every instance, two of the three titrations checked within one-tenth of 1 per cent.

Of a flume sample, 500 cu. cm. were first measured in a volumetric flask, noting the temperature of the same. This was then discharged into a separatory funnel for evaporation. This evaporation was conducted over an electric evaporator at a temperature lower than the boiling point. An electric fan blowing the water vapors away from the casserole increased the rate of evaporation about 40 per cent. The entire 500-cu. cm. sample was evaporated until only about 10 cu. cm. remained in the casserole. Care was taken to wash the separatory funnel with distilled water and evaporate this with the rest. Each evaporation required about 7 hours, depending somewhat on the humidity of the atmosphere. The sample was then taken to the burette for titration. The initial reading of the burette was taken after the tube had been filled with silver nitrate solution to within a few cubic centimeters of the top. One drop of potassium bichromate solution was added to the initial sample, and an additional drop was added for every 10 cu. cm. of silver nitrate solution used in the titration of the sample. The silver nitrate solution was added at the rate of about 4 drops per sec., until within a few cubic centimeters of the end point of the reaction. In the meantime, a glass stirring rod was rinsed with distilled water. The final quantity of silver nitrate solution was admitted very slowly, at the rate of a drop in about 2 sec., until a point was reached where the former lemon-yellow color of the

sample was permanently replaced by a very faint tint of orange. This being the end point sought, the burette reading was recorded. The difference between the initial and final reading, multiplied by 1 000 and divided by the actual volume, in cubic centimeters, discharged by the volumetric flask when the sample was measured for titration, gave the titration per liter of dosed flume water at the laboratory temperature.

The special dilutions were treated in a similar manner. Three dilutions were made for each experiment, in order to check the laboratory processes. The ratio for a given dilution approximated that which was obtained by dividing the weir discharge (computed by formula) by the salt rate, q. In some cases, this ratio could not be attained within several per cent. with the laboratory equipment at hand.

Each dilution was made by taking 10 cu. cm., as measured by a pipette, of the actual salt solution sample obtained during the experiment, and diluting it with the proper quantity of pond water obtained during the same test, until the prescribed ratio of dilution was obtained.

In almost every experiment it was necessary to make this dilution in two steps. For example, if a dilution in the ratio of 2 500 to 1 had been desired, it would have been made up as follows:

The contents of two 10-cu. cm. pipettes were discharged into a 500-cu. cm. volumetric flask, which had been washed previously with some of the pond water sample. The flask was then filled to the 500-cu. cm. mark with the pond water sample, and inverted forty times to secure a thorough mixture of the two solutions. The temperatures of both the salt solution and the pond water were recorded. This solution then had a ratio of dilution of approximately 25 to 1. The volume of one 10-cu. cm. pipette filled with this "stock" solution was discharged into a 1 000-cu. cm. volumetric flask, which had previously been washed with some of the normal pond water sample. The flask was then filled with the normal pond water up to the 1 000-cu. cm. mark and thoroughly shaken to insure a good mixture of the two solutions. The temperature of both the stock solution and the pond sample were recorded. This mixture then had a nominal ratio of dilution of 2 500 to 1. A 500-cu. cm. sample of this dilution was measured and treated in the same manner as the dosed flume water samples.

All precautions and directions advised by Mr. Groat in his paper on "Chemi-Hydrometry" were followed strictly, in order to obtain accurate dilutions.

Dilutions of the salt solution sample with distilled water were made in a similar manner. The ratio of dilution for each experiment was obtained from Fig. 20. The contents of a 10-cu. cm. pipette filled with the dilute solution was discharged directly into a casserole, and titrated in the same manner as the other two classes of samples after evaporation.

From start to finish, the whole experiment progressed as follows:

(*1*).—The mixing tank was filled with water and the required quantity of salt dissolved in it.

(*2*).—The flood-gates were adjusted to secure the proper head on the weir.

(*3*).—The water in the solution tank, in order to keep it soaked, so that its volume would be constant, was discharged. The salt solution was then passed from the mixing tank into the solution tank by gravity.

(*4*).—The experiment was then ready to begin. The first long whistle announced that the dosing had started.

(*5*).—A long bell signaled the arrival of the salt at the sampling station. The flume sampler started to flush his pipes and bottles, and the weir gauge readers placed themselves in readiness to read their instruments.

(*6*).—The second whistle announced the fact that the prescribed uniform rate of dosing had been obtained, and that the experiment had officially begun at the dosing end. The salt solution sample and the pond sample were started at this time.

(*7*).—Two short bells signaled the flume sampler to take his flume samples, and the hook-gauge readers to take their initial readings.

(*8*).—Bells every 20 sec. signaled simultaneous readings on all the weir gauges.

(*9*).—In the meantime, the assistant engineer proceeded to take the temperature of the salt solution and the flume water.

(*10*).—A third whistle designated the end of the experiment at the dosing station. The valve man discontinued the salt sample.

(*11*).—Three bells signaled the final gauge readings. The dosed flume water samples were discontinued and carried to the laboratory.

(*12*).—The chemist then proceeded to analyze the samples.

Record Forms.—Fig. 21 displays the heads of the different forms used in recording the data obtained for these experiments.

Calibrations and Corrections.

Stop-Watches.—The four stop-watches used in these tests were placed in the hands of a jeweler 2 months previous to the experiments, with instructions to operate and time them every day. When the experiments were started, all four watches were regulated within 1 sec. of error in 10 hours. However, subsequent regulation was found necessary, the stop-watches being timed each week during the progress of the experiments.

Two types of watches were represented: two continuous-running New England watches, and two Swiss speedway watches with cylinder movement. The New England watches gave much better satisfaction than either of those of Swiss make; but, in no experiment, was the stop-watch relied on absolutely in giving the duration of the test. The dosing recorder and also the timer kept accurate record of the time by observing the second hand on an ordinary watch. This could be read within a second. In most experiments, the three watches checked each other within a second; in such cases, the stop-watch record was used as the basis for computation.

Variation in Head on the Weir.—The experiments were performed during a period of comparatively high water in the Huron River. In no case did the pond elevation vary more than 0.05 ft. during the progress of the experiment. This insured a practically constant volume of water flowing through the flume.

The hook-gauge records, giving readings taken every 20 sec., show a maximum variation of head on the weir of 0.070 ft. for some high-head experiments. In these instances the water was traveling in the flume at a velocity exceeding 3.3 ft. per sec. With such a high velocity, the waves in the flume were of so great a magnitude that the floating railroad ties were ineffective in decreasing the amplitude of their motions. The variation decreases with a diminishing head on the weir, and, for heads below 3 ft., in no instance exceeds 0.025 ft. These variations are not due to a gradually increasing or diminishing head on the weir, but are either periodic or accidental.

The remarkable agreement between the mean readings for all three hook-gauges and the piezometer tube proves that "variation in head" was ineffective in producing error.

University of Michigan—Experimental Flume.

HOOK-GAUGE RECORD.

Observer_____ Date_____

Hook-Gauge Number_____ Correction of Datum_____

EXPERIMENT NUMBER.	TIME: Beg. and End	READING.	MEAN READING.	HEAD ON WEIR.

University of Michigan—Experimental Flume.

SOLUTION-TANK RECORD.

Observer_____ Date_____

EXPERIMENT NUMBER.	TIME: Beg. and End	SECONDS to Fall 0.1'	DURATION, (Sec.)	GAUGE READING.	DIFFERENCE.	VOLUME Cu. Ft.	RATE Cfs.

SPECIAL-DILUTION ANALYSIS.

EXPERIMENT NO.	STOCK DILUTION.							SAMPLE DILUTION.						ANALYSIS.			
	NOMINAL RATIO.	FLASK.		PIPETTE.		TEMP. SALT SOLUTION	TEMP. POND SAMPLE	NO. DILUTION	NOMINAL RATIO	FLASK NO.	PIPETTE NO.	TEMP. STOCK SOLUTION	TEMP. POND WATER	EVAP. COIL NO.	FLASK NO.	TEMP. OF SAMPLE	TITRATION OF $AgNO_3$
		NO. OF VOL'S USED	NUMBER	NO. OF VOL'S USED	NUMBER												

FLUME-WATER ANALYSIS.

DATE.	EXPER. NO.	SAMPLE NO.	EVAP. COIL NO.	FLASK NO.	TEMP. OF SAMPLE.	TITRATION.	MEAN TITRATION PER SAMPLE.	MEAN TITRATION FOR EXPER.

SALT-SOLUTION ANALYSIS.

DATE.	EXP. NO.	NOMINAL RATIO.	FLASK.		PIPETTE.		TEMP. SALT.	TEMP. H_2O	TEMP. STOCK.	PIPETTE NO.	TITRATION.
			VOL'S USED.	NO.	VOL'S USED.	NO.					

RECORD FORMS

FIG. 21.

Determination of the Width of the Weir Crest.—Table 1 gives the results of measurements of the width between the sides of the flume for different elevations above the weir crest. The value for the width of crest used in the computations is tabulated in the last column. From top to bottom, these values do not differ more than 0.06 of 1 per cent.

TABLE 1.—WIDTH OF WEIR CREST.

Height above weir, in feet.	Width of flume, in feet.	Width of weir crest, in feet.
0.00	6.560	6.560
0.25	6.556	6.558
0.50	6.564	6.560
0.75	6.560	6.560
1.00	6.548	6.558
1.25	6.561	6.558
1.50	6.559	6.558
1.75	6.558	6.558
2.00	6.555	6.558
2.25	6.548	6.557
2.50	6.555	6.557
2.75	6.555	6.557
3.00	6.550	6.556
3.25	6.550	6.556
3.50	6.553	6.556
3.75	6.560	6.556
4.00	6.556	6.556

Determination of the Elevation of the Weir Crest Relative to the Gauges.—In order to determine the elevation of the weir crest relative to the weir gauges, the water in the flume was lowered a fraction of an inch below the crest of the weir. A hook-gauge was then placed inside the flume, 1 ft. from the weir. By using a level, the reading of this hook-gauge when the point of the hook was at the elevation of the weir crest was ascertained and recorded. This was tested and checked for several points along the crest. The hook was then lowered to the water level, and simultaneous readings were made on this hook-gauge and those in the gauge shelter. A relation was thus established between the gauges and the weir. Two such tests were made by different observers, the results checking within 0.0001 ft. Actual levels run between the weir and the gauges checked the foregoing results within 0.001 ft. The elevation of Gauge No. 4 was established by levels run between Gauges Nos. 2 and 3. Readings from this gauge can be relied on within 0.001 ft.

Uniformity of the Mixture of the Salt Solution with Normal Water.—Table 2 shows how thoroughly the salt solution became mixed with the pond water as it passed through the flume. The distribution of

TABLE 2.—Titration, in Cubic Centimeters, of $AgNO_3$ Solution Required to Neutralize the Saline Content of 500-cu. cm. Samples of the Dosed Flume Water.

Experiment No.	Head on weir.	Sample Cock Number:				Mean.
		1.	2.	3.	4.	
1	0.393	58.65	58.15	.....	.....	58.40
2	0.486	49.84	49.45	.....	.....	49.65
3	0.680	49.41	49.75	.....	.....	49.58
4	0.991	43.97	43.40	.....	.....	43.69
5	1.093	51.93	52.00	.....	.....	51.97
6	1.203	48.78	48.75	.....	.....	48.76
7	1.309	48.73	48.65	48.72	.....	48.70
8	1.379	.....	55.69	55.71	.....	55.70
9	1.569	.....	51.28	51.63	.....	51.46
10	1.772	51.71	51.24	52.00	.....	51.65
11	1.866	.....	.....	56.17	.....	56.17
12	1.964	.....	47.55	47.50	.....	47.53
13	2.286	52.60	52.81	52.79	.....	52.73
14	2.386	51.42	51.11	51.27	.....	51.27
15	2.690	50.77	50.75	50.44	.....	50.65
16	2.791	54.20	53.30	52.64	.....	53.38
17	2.895	46.45	46.47	47.07	.....	46.66
18	3.231	50.05	48.97	49.24	.....	49.42
19	3.266	52.40	(50.63)	50.68	50.95	51.15
20	3.344	52.69	51.83	51.95	65.86	55.58
21	3.574	51.40	52.99	(52.70)	52.40	52.37
22	3.757	56.90	52.45	51.66	52.43	53.36
23	3.934	48.73	50.77	50.79	51.17	50.39

the mixture depended somewhat on the manner in which the flood-gates were opened. Better distribution of the salt solution could hardly be desired for all experiments up to 3.344 ft. of head. For higher heads, it was found necessary to lower the submerged weir, in order to secure the higher discharges. The resulting mixture was not as good as in the previous experiments, but only in Experiment 20 was it exceptionally bad. The excess of salt in the sample obtained from Cock No. 4 in this experiment is difficult to explain, and tends to make it less reliable by far than the others.

Calibration of the Solution Tank.—The metal calibrator was calibrated gravimetrically at different temperatures.

Starting with an empty solution tank, the contents of the calibrator were emptied into the solution tank from the spillway platform above. The solution tank gauge readings before and after emptying were recorded. The calibrator was then refilled, and the process was repeated successively until the tank was full.

Two calibrations of the solution tank were made with water at different temperatures at two different times during the progress of the experiment. These calibrations checked within 0.1 of 1 per cent.

A mean value of 1.374 cu. ft. for each 0.1 ft. of depth was used in all computations.

The solution tank was never left empty for any length of time, water being always maintained in the tank in order to insure constancy of volume. However, care was taken before each experiment to empty completely all the water from the tank and piezometer before admitting the salt solution. The tank was placed under the spillway platform and in this manner was protected at all times from the heat of the sun.

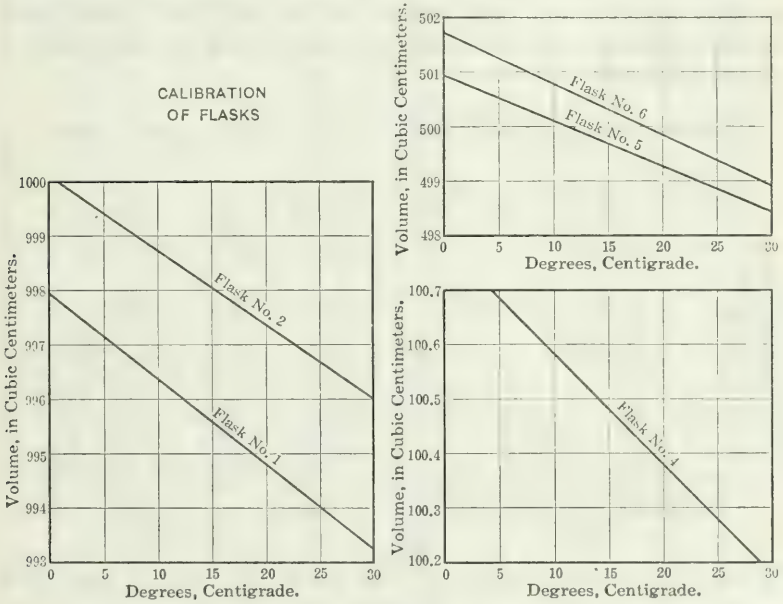

FIG. 22.

Calibration of Flasks and Pipettes.—The flasks and pipettes were carefully calibrated by R. A. Dodge, Teaching Assistant in Hydraulic Engineering, University of Michigan. Most of this work was conducted in the Physical Laboratories of the University, and the remainder on the balances at the flume.

The discharge volume of the pipettes was measured at different temperatures for different concentrations of salt solution. The flasks were calibrated at different temperatures only, using distilled water.

The results of these calibrations are represented graphically by the curves of Figs. 22 and 23. These curves were used in the final computations of the discharge by the salt solution method.

Low-Head Experiments.—For heads on the weir of less than 0.393 ft., with the apparatus at hand, it proved impossible to obtain reliable experiments, using the salt solution method. This was due to the fact that the mean flume velocity was less than 0.2 ft. per sec.; consequently, the mixing of the salt solution was poor, and the sprinkling system was not designed to meet the conditions present for so small a gate-opening.

Computation of Results.

Table 3, Plate II, gives the final computations on the data obtained in these experiments. Each step in the computation is made clear by the explanatory column at the left, designating the mathematical process involved.

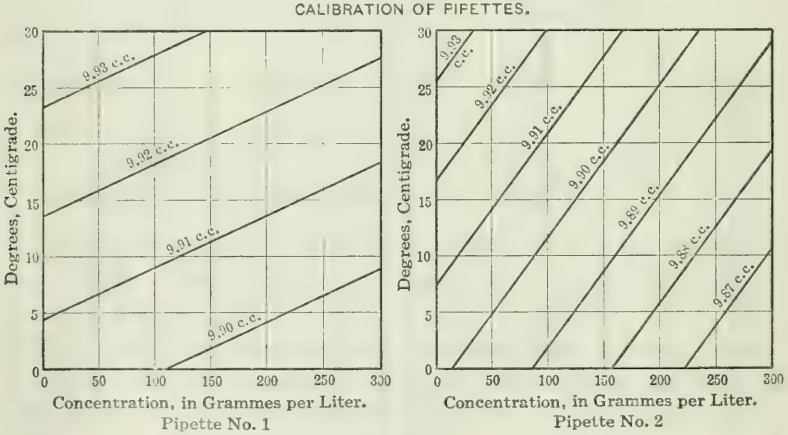

Fig. 23.

Computations were made in long hand. In general, they were carried out so that a change of one unit in the last significant figure produced a difference of less than 0.03 of 1% in the final result.

The Theiesen, Scheel, and Diesselhorst tables for distilled water, and the Landolt and Börnstein tables for solutions of sodium chloride, were used in all computations. The Williams and Hazen hydraulic tables were used in checking the weir discharge, as computed by the Bazin formula.

The shrinkage coefficient, k, did not depart far enough from unity to change the value of the last significant figure in any of the experiments. It will be noted that it has been tabulated equal to 1.0, although, for some experiments, its actual value was 0.971.

PLATE II.
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TABLE 3.—DATA AND COMPUTATIONS.

Line.	Formula.	Unit.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	DATE OF EXPERIMENT, 1917.		April 26.	April 26.	April 27.	April 27.	April 27.	April 27.	April 30.	March 27.	March 27.	March 26.	March 26.	March 24.	March 29.	March 29.	March 30.	March 31.	April 3.	April 4.	May 2	April 4	April 5.	April 6.	April 7.
	TEMPERATURES.																								
2	Temperature — Salt solution in tank	°C	8.0	7.0	8.0	11.2	10.5	13.0	13.5	4.0	4.0	7.0	7.0	4.0	3.0	3.0	3.0	4.0	4.0	6.5	5.0	3.5	3.5	3.0
3	Salt solution when pipetted for sample	°C	19.5	19.3	19.2	19.2	19.2	19.5	19.2	19.5	19.5	6.5	16.7	14.3	13.1	16.2	14.8	15.0	13.8	15.0	15.0	10.7	15.5	13.5	11.8
4	Salt solution when pipetted for special dilutions	°C	20.1	19.5	19.5	19.5	19.5	19.5	19.5	19.5	19.5	6.6	17.0	15.1	14.1	17.2	15.8	15.8	14.8	15.8	15.8	11.7	15.9	14.5	12.6
5	Distilled water when mixed with salt solution	°C	21.5	21.5	20.5	19.9	19.8	19.8	19.8	19.8	19.8	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0
6	Normal tap water when mixed with salt solution	°C	21.5	21.5	19.5	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0
7	Salt solution in dilution when pipetted for titration	°C	19.5	19.5	19.5	19.5	19.5	19.5	19.5	11.5	12.5	12.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
8	Stock dilution when pipetted for further dilution	°C			19.5	19.5	19.5	19.5	19.5	11.5	12.5	12.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
9	Stock solution when pipetted for titrating stock solution	°C			19.5	19.5	19.5	19.5	19.5	11.5	12.5	12.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
10	Average temperature of special dilutions and samples	°C	20.5	20.4	19.9	19.6	19.6	19.6	19.6	14.0	17.4	9.5	16.8	7.0	14.7	11.0	16.9	10.5	15.0	10.9	7.0	10.6	16.5	11.0	19.4
11	Average temperature of dosed flume-water samples	°C	18.9	19.3	19.5	19.2	19.2	19.2	19.2	14.0	17.0	9.5	16.8	7.0	14.7	11.0	16.9	10.5	15.0	10.9	7.0	10.6	16.5	11.0	19.4
12	Temperature of flume water during the test	°C	8.0	8.0	9.0	11.2	11.2	11.5	12.5	12.5	12.7	10.8	11.7	10.4	12.8	8.0	10.4	11.5	11.0	11.0	6.0	6.5	9.0	7.0	9.0
	SALT SOLUTION ANALYSIS.																								
13	Pipette used for salt solution sample in Line 3		14	14	1	1	1	1	3	3	3	3	3	3	3	3	3	1	1	1	1	1	1	1	1
14	Number of volumes discharged by pipette in Line 13		100	100	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
15	Total volume of salt solution pipetted at temperature of Line 3	c.c.	100 286	100.154	69 535	69 433	69 419	69 490	69.434	49.515	50.024	68.505	69.707	70.713	49.490	5.898	9 918	9.912	9.912	10.727	10 918	10 940	9 721	9 918	9 909	9 905
16	Flasks used in measuring the distilled water dilution		4	4	1	1	1	1	1	1	1	1 000 3	999.0	999.0	1 998 8	800.8	1	1	1	1	1	1	1	1	1
17	Volume of flasks in Line 16 at temperature of Line 3	c.c.	4005	4 000	1 000	999.0	998.8	998.8	998.8	998.8	998.8	1 000.3	999.0	999.0	1 998.8	800.8	1 000.4	1 000.4	1 000.4	1 000.4	1 000.4	1 000.4	1 000.4	1 000.4	1 000.4
18	Dilute salt solution discharged by pipette at temperature of Line 7	c.c.	49 93	49.93	49.50	49.63	47.50	44.56	50.00	49.79	47.76	47.50	41.40	43.70	49.81	44.30	49.54	47.08	48.30	47.42	52.06	0.915	0.921	0.921	0.918
19	Titration per litre of dilute salt solution	c.c.	5 043	4477	4 563	3 990	4 016	4 498	4 511	4 512	4 617	4 754	4 474	4 668	5 408	4 964	5 379	5 043	4 764	4 850	5 745	4 640	4 650	4 650	4 650
20	Titration per litre of dilute salt solution at laboratory temperature		5 709	5 579	5 192	14 27	14 38	14 74	14.76	20.19	20 15	5 381	5 351	33 63	47.85	50 54	50 47	50 42	57 05	75 45	73 44	5 255	5 255	5 255	5 255
21	1 000 c.c. Line 15 = Line 19 / Line 17 = Line 20		13 070	90 270	32 930	57 410	69 120	75 950	58 83	84 420	101 120	191 620	144 310	150 370	154 630	317 970	325 710	271 600	327 670	290 000	261 500	350 760	457 000	351 500	349 779
22	Titration per litre of salt solution in Line 5		9 72	10 48	10 99	39 63	35 50	33 83	43.56	56 39	68 78	74 33	77 59	79 90	118 47	215 47	199 59	150 97	186 70	204 09	243 70	9.70	896 99	254 56	349 13
23	Concentration of salt solution at temperature of Line 5	g/L	9 72	10 48	10 99	39 63	35 50	33 83	43.56	56 39	68 78	74 33	77 59	79 90	118 47	215 47	199 59	150 97	186 70	204 09	243 70	9.70	896 99	254 56	349 13
24	Approximate percentage of salt solution at temperature of Line 3	%	1.0	1.0	1.0	3.9	3.5	3.3	4.3	5.6	6.8	7.4	7.8	8.0	11.8	21.5	19.9	15.0	18.6	20.4	24.3	1.0	89.7	25.4	34.9
25	Density of salt solution to tank, at temperature of Line 3		1.0072	1.0073	1.0079	1.0311	1.0262	1.0229	1.0252	1.0292	1.0296	1.0346	1.0346	1.0349	1.0388	1.0540	1.0527	1.0478	1.0513	1.0493	1.0491	1.0140	1.1377	1.1213	1.1758
26	Density of salt solution at temperature in Line 5		1.0047	1.0052	1.0063	1.0261	1.0224	1.0211	1.0243	1.0284	1.0284	1.0333	1.0333	1.0333	1.0361	1.0501	1.0491	1.0443	1.0481	1.0461	1.0461	1.0101	1.1337	1.1203	1.1651
27	Density of salt solution at temperature in Line 4		1.0072	1.0078	1.0079	1.0319	1.0277	1.0254	1.0291	1.0295	1.0295	1.0375	1.0433	1.0391	1.0433	1.0555	1.0545	1.0495	1.0533	1.0513	1.0514	1.0201	1.1428	1.1325	1.1863
28	Density of salt solution at temperature in Line 5		1.0072	1.0078	1.0079	1.0319	1.0277	1.0254	1.0291	1.0295	1.0295	1.0375	1.0433	1.0391	1.0433	1.0555	1.0545	1.0495	1.0533	1.0513	1.0514	1.0201	1.1428	1.1325	1.1863
29	Titration per litre of salt solution at temperature in Line 19		18 100	20 300	33 000	57 430	69 110	75 520	58 370	84 370	101 320	191 190	144 170	150 750	155 900	318 750	326 250	271 600	328 400	290 540	262 810	350 140	459 000	351 000	349 140
	DISCHARGE OF SALT SOLUTION.																								
30	Tank gauge-board reading at beginning of experiment	ft.	0 900	0.800	0 950	0.700	0.700	0.900	0.800	0.700	0.400	0 800	0.700	0.900	0.800	0.800	0 800	0.600	0.600	0.750	0 800	0 700	0 800	1 000	1 000
31	Tank gauge-board reading at end of experiment	ft.	0 200	0 300	0 800	0 300	0 300	0 500	0 300	0 300	0 300	0 600	0 300	0 300	0 300	0 300	0 350	0 300	0 300	0 300	0 300	0 300	0 100	0 700	0 300
32	Volume of salt solution discharged at temperature of Line 5	cu ft	51 008	37 978	29 978	32 005	30 698	28 278	27 978	28 003	28 003	30 698	30 698	30 698	30 698	32 399	30 698	30 698	30 698	30 698	30 698	51 008	51 008	51 008	30 220
33	Time of discharge	sec.	959 0	974 5	948 7	861 0	887 0	956 9	946.0	923 4	922.0	980.0	946 0	955.0	922 8	955.0	926 5	922 9	981.4	869.0	929 0	964 0	957 502	881 502	870 502
34	Rate of discharge of salt solution at temperature of Line 19	c.f.s.	0.05313	0.03898	0.03163	0.03718	0.03467	0.02956	0.02984	0.03045	0.03037	0.03055	0.03251	0.03199	0.03329	0.03393	0.03477	0.03476	0.03128	0.03529	0.03103	0.05319	0.05423	0.03855	0.03578
35	Rate of discharge of salt solution at temperature of Line 19		0.04413	0.03266	0.03429	0.03463	0.03467	0.03046	0.03084	0.03045	0.03035	0.03055	0.03211	0.03349	0.03500	0.03477	0.03497	0.03628	0.03471	0.03480	0.03404	0.05319	0.05423	0.03855	0.03578
	SPECIAL DILUTION ANALYSIS.																								
36	Volume of salt solution discharged by pipette at temperature of Line 4	c.c.	9 980	9.961	9.930	9.917	9.930	9.901	10.026	9.977	9.990	9.971	9.966	9.964	9.914	9.913	9.913	9.914	9.914	9.913	9.904	9.927	9.916	9.935	9.913
37	Time of temperature in Line 36	sec.	9 005	9.004	9.013	9.014	9.027	9.013	9.010	9.008	9.008	9.008	9.008	9.004	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000
38	Numbers of flasks used in making stock dilution		9, 6, 4	9, 6	4	2, 6	6	6	6	Flask No. 4	6	6	5, 4	1, 5	5	1, 6	1, 6	1, 6	1, 6	1, 6	1, 6	1, 6	1, 6	1, 6	1, 6
39	Number of pipette volumes used		4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
40	Volume of dilution at temperature of Line 6	c.c.	1 997 1	2 493 1	2 484 1	501.5	499.0	498 0	500.0	500.0	500.4	997.8	750.0	1 494.0	500.0	501.0	1 495.5	1 494.4	996.1	1 495.5	1 494.4	996.1	1 495.5	1 494.4	400.7
41	Time of discharge of dilution at temperature of Line 6 to Line 19	sec.	1 997 0	2 493 0	2 484 1.0008	501.5	499.0	498.0	500.0	500.0	500.4	997.8	750.0	1 494.0	500.0	501.0	1 495.5	1 494.4	996.1	1 495.5	1 494.4	996.1	1 495.5	1 494.4	400.7
42	Volume of dilution at temperature in Line 19		1 998 0	2 495 2	2 487 0	501.9	499.4	498.4	500.4	500.4	500.8	998.6	750.7	1 495.4	500.4	501.4	1 497.0	1 495.9	997.5	1 497.0	1 495.9	997.5	1 497.0	1 495.9	401.2
43	Rate of dilution at temperature in Line 19	c.c.	150 99	251 85	251 96	6.3105	7 394	6.490	9.935	10.327	12 856	16.778	17 667	18 825	23.201	90 385	89 552	90.279	89.593	91.513	91.832	90.385	89.552	90.279	89.593
44	Rate of stock dilution discharged by pipette at temperature of Line 8					9 919	9 934	9 918	9.928	9.933	9.931	9.915	9.917	9.915	9.905	9.914	9.918	9.914	9.911	9.912	9.940	9.914	9.918	9.914	9.911
45	Time of discharge of stock dilution at temperature of Land 8					0 9997	0 9994	0 9999	0.9995	1.0001	0 9997	0 9995	0 9997	0 9995	0 9990	0 9997	0 9991	0 9994	0 9997	0 9993	0 9990	0 9997	0 9991	0 9994	0 9997
46	Rate of stock dilution pipetted at temperature of Line 9					9 915	9 938	9 913	9.923	9.928	9.926	9.915	9.919	9.916	9.914	9 918	9 913	9 913	9 913	9 913	9 943	9 913	9 913	9 913	9 913
47	Volume of flask measuring final dilution at temperature of Line 9	c.c.				998.8	997 8	998.0	998.5	998.8	997.8	998.8	997.9	998.8	998.0	998.4	998.0	998.6	998.8	998.6	998.8	998.6	998.8	998.6	998.8
48	Volume of flask at temperature in Line 22	c.c.	1 001	1 001	1 001	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9	998.9
49	Ratio of dilution at final dilution			Line 48 = Line 48		100.55	100.50	100.39	100.40	100.71	100.50	100.04	100.31	100.43	100.34	100.49	100.36	100.49	100.30	100.49	100.33	100.13	100.30	100.13	100.16
50	Total ratio of dilution for special dilutions		181 29	Line 43 = Line 49	251.85	501 58	501.0	501.0	501.0	501.0	501.0	1 671.1	1 771.2	1 501.0	501.0	501.0	501.0	501.0	501.0	501.0	501.0	4 997.0	5 000.0	5 000.0	5 000.7
51	Value of shrinkage-coefficient		1.002		1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002	1.002
52	Value of ratio of titrations		191.02	253 32	253.96	504.16	504.0	503.9	504.0	1 077.0	1 203.9	1 697.8	1 779.3	1 510.0	503.8	5 044.0	5 035.0	5 035.0	5 035.0	5 035.0	5 035.0	5 002.7	5 005.0	5 005.0	5 007.7
53	Flask measuring special dilution for evaporation		1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
54	Titration per litre of special dilution	c.c.	190.7	250.9	252.8	499.8	499.0	498.8	499.5	500.7	500.1	600.4	561.9	662.4	500.0	500.0	500.0	500.0	500.0	500.0	500.0	499.0	499.5	500.1	499.3
55	Titration per sample at temperature of Line 3	c.c.	119.06	43.45	43.97	87.18	87.50	86.97	87.55	88.78	88.55	87.97	87.19	87.60	86.90	87.15	87.45	87.15	87.50	87.54	87.54	87.15	87.54	87.54	87.54
56	Titration per litre at temperature in Line 10		119.06	91.17	108.04	87.18	106.56	96.31	96.33	96.31	96.33	96.66	106.89		96.90	100.23	99.07	100.09		94.24	108.04	104.44	105.12	96.56	106.90
57	Ratio of titrations at temperature of Lines 10 to 19	c.c.	1.0010	1.0021	1.0008	1.0010	1.0010	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0008	1.0005	1.0008	1.0008	1.0004
58	Titration per litre at temperature in Line 19	c.c.	119.18	91 91	108.16	87.30	106 56	96 42	96.33	96.42	106.49	106 17	96.66	106.61	96.94	100.33	99.07	100.09	104.44	94.31	108.04	104.44	105.12	96.65	106.99
	DOSED FLUME-WATER ANALYSIS.																								
59	Flask measuring flume water for evaporation		5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
60	Volume of flask at temperature of Line 11	c.c.	499 8	499.8	499.8	500.6	500.6	500.6	500.6	500.9	500.9	500.0	499.9	500.5	499.9	500.5	500.0	500.0	500.0	500.0	500.0	499.7	499.8	500.0	499.9
61	Titration per sample at temperature in Line 11		68 40	49.65	49.59	48.69	48.69	48.70	48.70	48.70	48.70	51.45	51.55	66.17	47.53	50.78	50.65	50.65	50.65	49.48	51.15	50.55	50.39	50.39	50.39
62	Titration per litre at temperature in Line 11		136 94	99.31	99.18	97.30	97.38	97.41	97.41	97.22	97.22	102.90	103.10	132.34	95.06	101.56	101.30	101.30	101.30	98.96	102.30	101.10	100.78	100.78	100.78
63	Ratio of dilution at temperature of Lines 11 to 19		1.1501	1.0915	1.0916	1.0021	1.0021	1.0026	0.9926	1.0000	1.0000	1.0000	1.0008	1.0008	1.0008	1.0008	1.0008	1.0004	1.0008	1.0008	1.0008	1.0007	1.0007	1.0004	1.0005
64	Titration per litre at temperature in Line 19	c.c.	157.39	108.39	108.25	97.57	104.08	97.52	96.42	111.45	108.56	106.33	103.18	118.48	95.20	103.33	103.23	102.39	106.32	98.15	104.65	106.79	105.51	104.85	106.95
	COMPUTATION OF DISCHARGE.																								
65	Line 54 — Line 56	c.c.	−3 09	+8.31	−2 60	+0.07	−1 64	−1.87	+3.10	+1.97	−2.14	+4.42	+6.01	−0.17	+3 61	+3 83	+1.98	−2.95	−0.96	−4.17	−1.56	−2 87	−4 19	−2.14	−2 18
66	Line 61 × Line 54		100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	110	110	110	110
67	Line 66 — Line 62		17 940	20 270	32 950	87 840	69 440	75 130	58 270	84 270	101 110	191 680	144 670	150 810	155 150	318 650	326 180	272 800	328 350	290 650	262 910	350 250	459 040	351 050	349 250
68	Line 66 × Line 60		684 2	496 6	496 0	444 6	444 6	1 847 4	1 370 0	1 370 0	1 370 0	2 894 8	2 894 8	10 563	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8	2 894 8
69	Line 68 — Line 60		−0 0277	+0.1034	−0 0227	+0.0008	−0.0178	−0 0004	+0 0163	+0 0147	−0.0023	+0 0547	+0.0408	−0 0031	+0 0379	+0 0483	+0 0336	−0.0041	−0.0010	−0 0433	−0.0158	−0 0297	−0 0434	−0.0221	−0 0225
70	Line 67 × Line 69		497 1	2096 9	1 160 9	0 7043	635 8	138.8	958 3	1 386 0	232 9	10 485	5 900	330 9	448 9	1 539	1 094	1 128	950 5	1 258	761 1	1 039	1 992	773 1	786 5
71	Line 58 = Line 70		164 5	2096 9	1 160 9	633 8	735 0	892 3	970 3	977 4	1 291 0	1 540 8	1 656 0	1 894 0	1 503 8	2 481 2	2 971 2	2 114 7	2 479 6	2 194 10	2 524 2	2 481 2	2 971 2	2 114 7	2 479 6
72	Line 63 × Line 71	c.f.s.	0 258	7 66	4 16	2 27	2 65	3 21	3 49	3 52	4 65	5 54	5 96	6 82	5 41	8 93	10 69	7 61	8 93	7 90	9 08	8 93	10 69	7 61	8 93
73	Line 59 × Line 63	c.f.s.	0 08	0 18	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08	0 08
74	Discharge by salt solution method: Line 72 + Line 73	c.f.s.	0 33	7 71	4 24	2 35	2 73	3 29	3 57	3 60	4 73	5 62	6 04	6 90	5 49	9 01	10 77	7 69	9 01	7 98	9 16	9 01	10 77	7 69	9 01
	WEIR OBSERVATIONS AND COMPUTATIONS.																								
75	Average head by Gauge No. 3—Sadd gauge	ft.	0 826	0 496	0 620	0 091	0 098	0 128	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109
76	Average head by Gauge No. 1—Farley and Stearns gauge	ft.	0 825	0 495	0 619	0 090	0 098	0 128	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109
77	Average head by Gauge No. 2—Franke gauge	ft.	0 826	0 495	0 619	0 091	0 098	0 128	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109
78	Average head by piezometer tube	ft.	0 826	0 495	0 619	0 091	0 098	0 128	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109	0 109
79	Weir discharge by Basin formula: $\left(0.405 + \frac{0.003}{h}\right)\left[3.1 c\, b\, h^{\frac{3}{2}}\left(1 + \frac{h}{d}\right)^{\frac{2}{3}}\right]$	c.f.s.	0 58	7.66	10 58	2 26	2 65	3 24	3 46	3 50	4 59	5 48	5 91	6 81	5 35	8 85	10 56	7 53	8 85	7 84	8 98	8 85	10 57	7 53	8 85
80	Line 74 — Line 79	c.f.s.	−0 25	+0 09	+0 07	−0 05	+0 09	+0 05	+0 11	−0 09	−0 04	+0 04	−0 17	+0 24	+0 45	+0 97	+0 97	−0 84	+0 14	−0 13	−1 37	+1 81	−0 51	−1 09	−0 19
81	Percentage of difference	%	−7 58	+1 17	+0 66	−2 23	+3 45	+1 55	+3 18	−2 50	−0 88	+0 75	−2 82	+3 48	+8 36	+10 77	+9 00	−10 94	+1 57	−1 65	−14 96	+20 11	−4 73	−13 92	−2 11

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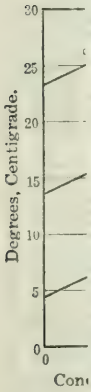

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Probable Error in the Measurement of the Discharge by the Salt Solution Gaugings.

In every process of calibration, observation, and experimentation, it was sought to eliminate errors greater than 0.1 of 1 per cent. The reliability of the result for any experiment, therefore, is probably best indicated by the excellency of the distribution of the salt content of the differently dosed flume-water samples. This has been discussed, and is exhibited in Table 2.

There is no doubt that some experiments give the correct discharge within 0.1 of 1 per cent. In general, it might be said that the maximum variation from the true discharge is less than 0.6 of 1% for every experiment except No. 20. This may even be claimed for the experiments for which some of the samples are missing, due to the good distribution of the saline content for these discharges. The distribution of the salt in Experiment No. 20 is so bad that the resultant discharge can hardly be relied on within 2 per cent.

The arithmetical mean of the quantities of silver nitrate solution required to titrate the different samples was used as the value of the silver nitrate titration for the entire section. With the excellent salt and velocity distribution in most experiments, this introduced only a negligible error. In fact, due to the equal spacing of the sampling pipe perforations, the arithmetical mean was automatically taken in a horizontal direction; hence, it is doubtful whether the refinement of weighting the samples in a vertical direction is warranted.

Greater accuracy in the chemical gauging could have been attained, had it not been necessary to resort to the evaporation of the samples in making the chlorine analysis. However, the use of the direct method of analysis was prohibitive, for financial reasons, as it would have necessitated putting in a dosing apparatus of larger capacity and also the purchase of a much greater quantity of salt.

The silver nitrate solution used in the titration was as weak as it could be made and still produce accurate results; hence, further weakening of this reagent, in order to reduce the quantity of salt required in a "dosed water sample", was prohibitive. Although it is true that the experiments on the very lowest heads could have been performed with a saturated solution of salt in the existing apparatus, dosing at such a rate that the samples of dosed flume water would

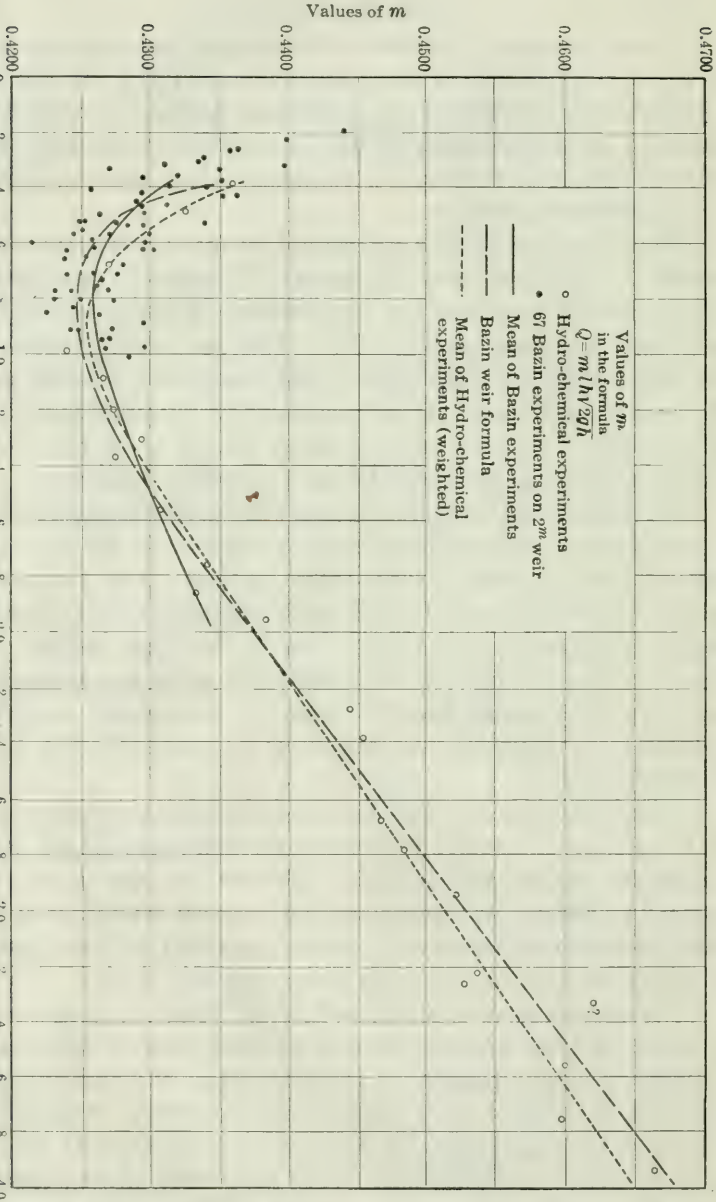
Values of m
0.4700
0.4600
0.4500
0.4400
0.4300
0.4200
Values of m in the formula $Q = m\,l\,h\sqrt{2gh}$
Hydro-chemical experiments
67 Bazin experiments on 2m weir
Mean of Bazin experiments
Bazin weir formula
Mean of Hydro-chemical experiments (weighted)
0 .2 .4 .6 .8 1.0 .2 .4 .6 .8 2.0 .2 .4 .6 .8 3.0 .2 .4 .6 .8 4.0
Head on Weir, in Feet.

FIG. 24.

require no evaporation before analysis, this could hardly be done for heads in excess of 0.4 ft. on the weir.

The ratio of evaporation used was about 50 to 1; that of dilution was about 6 000 to 1. Direct analysis of the dosed sample, therefore, would require a dilution ratio of approximately 120 to 1, with the same concentration of salt solution for each experiment as was used in these tests. Had the solution tank been of fifty times the capacity of the existing tank, the experiment on the highest head (4 ft. on the weir) could have been executed so as to necessitate no evaporation of the dosed flume water samples; but it would have required the solution of at least 20 tons of salt for the single experiment. This would be about three times as much salt as was actually used in performing the entire set of experiments. As has been previously mentioned, 50 bbl. of salt were used in these tests, costing $1.43 per bbl. Direct analysis would have required $50 \times 50 = 2\,500$ bbl., the cost of which is sufficient reason for resorting to evaporation at the sacrifice of a small degree of accuracy.

Conclusions.

Fig. 24 presents a graphical comparison of these experiments with those of Bazin on a similar weir, the Bazin experimental curve, and the Bazin formula. The weighted mean resulting curve of these experiments is shown by the dotted line. From this comparison, the following conclusions are evident:

(*1*).—For heads of less than 2 ft., the discharges, as given by the salt solution gaugings, are higher than those computed by the Bazin weir formula.

(*2*).—For heads greater than 2 ft., the Bazin formula gives discharges which are greater than those obtained in this set of experiments. This attains a maximum difference of 0.6 of 1% at a head of 4 ft. This fact is proved beyond doubt. In the future, engineers should make this correction in computations on high heads by the Bazin formula.

(*3*).—The salt solution experiments which lie within the range of the Bazin experiments, coincide with the Bazin formula and experimental curve more closely than the Bazin experiments themselves. The 67 experiments performed by Bazin on a weir 2 m. wide and 3.72 ft. high are shown by the small dots on Fig. 24.

(4).—The results of the high-head experiments fit the Bazin formula more closely than they would fit the probable extension of the Bazin experimental curve.

(5).—In claiming that the Bazin formula gives actual discharge within 1%, within the range of his experiments, this formula is all that its author maintains. There is, however, further merit for the Bazin formula, in that this statement proves to be valid for heads as high as 4 ft., on a weir 3.72 ft. high. The Bazin weir formula is thus extended 100% beyond its original legitimate field of application.

(6).—Table 3, Plate II, reveals the fact that simultaneous gauge readings on similar gauges, with similar taps, but applied at different points above the weir, do not differ by more than 0.005 ft. in extreme cases. In some experiments the readings even coincide, and, on the whole, there is no evidence of a tendency toward a definite positive or negative difference one from the other. Such differences as were actually observed might be attributed to accidental error and to the personal error of the different observers. Certainly the fact that the conducting pipe and tapping hole were 2 in. in diameter eliminated all error as to the nature of the tap.

The points at which these gauges tapped the flume duplicated the experimental points of observation used by Bazin, Fteley and Stearns, and Francis. Hence, these experiments seem to indicate that the discrepancies between the Bazin, Francis, and Fteley and Stearns formulas cannot be accounted for by the different points of observation used in their experiments. If the difference is due to the method of tapping, it must be accounted for in the nature of the tap rather than its location.

(7).—Finally, Table 4 presents a comparison between the results of these experiments and the following existing weir formulas:

Bazin, $Q = \left(0.405 + \frac{0.00984}{H}\right)\left[1 + 0.55\,\frac{h^2}{(p+h)^2}\right] L\,H\,\sqrt{2\,g\,H}$.

Fteley and Stearns, $Q = 3.31\,L\,H^{\frac{3}{2}} + 0.007\,L;\ H = h + 1.5\,\frac{v^2}{2\,g}$.

Francis, $Q = 3.33\,L\left[(D+h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right]$.

H. W. King, $Q = 3.34\left(1 + 0.56\,\frac{H^2}{(p+H)^2}\right) L\,H^{1.47}$.

TABLE 4.—Discharge per Foot of Weir Crest.

Head on weir.	Chemical.	Bazin.		Bazin.		Fteley and Stearns.		Francis.		King.	
	Experiment, in cubic feet per second.	Experiment, in cubic feet per second.	Percentage of error.	Formula, in cubic feet per second.	Percentage of error.	Formula, in cubic feet per second.	Percentage of error.	Formula, in cubic feet per second.	Percentage of error.	Formula, in cubic feet per second.	Percentage of error.
0.40	0.884	0.874	1.13	0.876	− 0.90	0.848	− 4.07	0.845	− 4.41	0.873	− 1.24
0.60	1.599	1.591	− 0.50	1.588	− 0.69	1.557	− 2.63	1.559	− 2.50	1.593	− 0.38
0.80	2.445	2.444	0.04	2.437	0.33	2.407	− 1.56	2.401	− 1.80	2.448	+ 0.12
1.00	3.416	3.423	+ 0.21	3.410	− 0.18	3.370	− 1.35	3.367	− 1.44	3.424	+ 0.23
1.20	4.506	4.511	+ 0.11	4.499	− 0.16	4.467	− 0.87	4.442	− 1.42	4.512	+ 0.13
1.40	5.709	5.706	0.05	5.699	− 0.18	5.661	− 0.84	5.618	− 1.59	5.705	− 0.07
1.70	7.709	.....	.....	7.698	− 0.14	7.650	− 0.77	7.568	− 1.83	7.688	− 0.27
2.00	9.928	.....	.....	9.923	− 0.05	9.865	− 0.63	9.704	− 2.26	9.886	− 0.42
3.00	18.806	.....	.....	18.881	+ 0.40	18.806	0.00	18.160	− 3.43	18.667	− 0.74
4.00	29.833	.....	.....	30.006	+ 0.58	30.041	+ 0.70	28.416	− 4.75	29.485	− 1.17

The columns of Table 4 denoting the percentage of difference between these hydro-chemical gaugings and the other formulas, lead to the following conclusions:

(1).—The agreement with the Bazin experimental curve is remarkably good, except for the low heads.

(2).—The Bazin formula approximates these experiments within 1% for all discharges listed.

(3).—The Fteley and Stearns formula gives good results for the highest heads, but gives discharges too small for heads of less than 1 ft.

(4).—The Francis formula does not agree with these experiments. The discharges vary from 1.4 to 4.75%, and are lower than the hydro-chemical experiments, for both low and high heads on the weir.

(5).—The King weir formula approximates these experiments very closely for nearly all heads; in fact, it seems to fit the data better than any of the existing formulas, except for a divergence of $1\frac{1}{4}$% at the lowest head.

Acknowledgments.

The writer is indebted to H. W. King, M. Am. Soc. C. E., Professor of Hydraulic Engineering, University of Michigan, for his assistance and co-operation in making these experiments possible. His interest in the experiments was a continual inspiration, and his suggestions were responsible for many improvements in the manner of conducting them. The writer also wishes to express his appreciation for the invaluable services of R. A. Dodge, Teaching Assistant in Hydraulic Engineering, University of Michigan. The paper by Mr. Groat on "Chemi-Hydrometry" was a constant source of reference in the execution of these experiments. Having used the methods described in this paper very freely, the writer wishes to acknowledge his indebtedness to Mr. Groat. For assistance in the operation of the experiments, the writer wishes to thank Mr. C. O. Wisler,* Instructor in Civil Engineering, University of Michigan, and the following senior and graduate civil engineering students at that University in 1917: Messrs. O. B. Apesteche,† C. E. Gill, E. R. MacLaughlin, S. P. Davis, T. G. Bedford, G. A. Covarrubias, J. B. Jewell, H. J. McFarlan, E. Gusman, T. B. Dimmick, A. H. Cohn, A. B. Pastor, J. W. Van Brunt, C. E. Wells, M. F. Wagnitz, J. B. Franks,* G. A. Pomeroy, and A. Marino.

* Now Assoc. M. Am. Soc. C. E.

† Now Jun. Am. Soc. C. E.

DISCUSSION

Mr. Herschel.

CLEMENS HERSCHEL,* PAST-PRESIDENT, AM. SOC. C. E. (by letter). —Few engineers are in a position to appreciate from personal experience the amount of work involved in making the experiments necessary for writing a paper such as this, and in writing the paper itself. In every way such contributions should be encouraged, as they form steps in the advancement of engineering knowledge.

One criticism only the writer allows himself, and that relates merely to the title of the paper: In a strict sense, hydro-chemical gaugings are not competent to "verify" a weir formula. That can only be done by a tank, or volumetric gauging. The hydro-chemical method itself needs verification, by comparison with tank gaugings.

Still, an approximation to such a verification can be made, and is made when able engineers, like Mr. Groat, have first compared hydro-chemical results with those produced by weir gaugings made under their direction, themselves founded on comparisons with tank gaugings.

What the paper really presents is a "comparison" between the results of the Bazin and other weir formulas, and those deduced from hydro-chemical gaugings; both methods being in the same degree in need of verification.

And, if one pleases, even a tank gauging is not absolute. It is, for example, impossible to make a rod exactly 1 ft. long. Still less possible to mark out exactly 1 sq. ft.; less possible again, to make a cube of which each edge shall measure exactly 1 ft.; and still less possible to gauge cubic feet per second.

The title of the paper, however, does not recognize, and is likely to mislead the reader as to, distinctions of the several sorts mentioned; of value in practice and in study, even though some of them be, if one likes, of the second degree of differentiation.

With the inclusion of the discussion by Frederic P. Stearns, Past-President, Am. Soc. C. E., Mr. Nagler's paper bids fair to become as important a paper on hydraulic engineering, as has appeared in recent volumes of *Transactions*; and the author may conclude that he "has builded wiser than he knew." The very scope and title of the paper, the "verification" of a weir gauging, by a complicated process, such as a hydro-chemical gauging, no doubt caused that old-time fetish of hydraulic engineers—weir gaugings, or the "standard weir"—to totter on its pedestal; while the plain-spoken words of Mr. Stearns: "(1) That in the present state of the art the weir is not an accurate instrument for the measurement of water", and the proofs supplied by the searching analysis that follows, must have caused that old idol to fall

* New York City.

Mr. Herschel. irretrievably from the high estate it has occupied in the minds of a numerous body of engineers, lo, these many years.

Under the circumstances, the writer has concluded that it might be of interest to hydraulic engineers were he to extend the remarks he has already made in discussion of Mr. Nagler's paper.

If one takes Fig. 26 of Mr. Stearns' discussion, it must be evident, on reflection, that an apparatus of this kind, carrying water which has a material velocity of approach, cannot cause that water to spill over the crest of the weir with a regular, or devoid-of-turbulence, motion of its particles. The velocity of approach must be changed in direction, upward and "over the top" (to use a prevalent expression of the day). If, as in truth, this velocity of approach is only the resultant in one direction of turbulent motion in all sorts of directions, but prevailing toward the weir, then such turbulence will necessarily be increased in the angle between the up-stream face of the weir and the bottom of the flume. The elements of turbulent motion are largely accidental, and hence do not lend themselves to determinations of precision.

Again, why the seemingly excruciating process of dragging the particles of water across the sharp edge of a metal right-angled corner of a bar of iron; which torture the water resists, and it protests and squirms, until it jumps upward and "over the top", in defiance of its own inclinations and the laws of gravity, so that it is forced to form a little hollow next down stream from the weir crest?

The question may be answered categorically by pointing out that 200 years ago, or more, when hydraulic experiments were in their infancy, the idea of a discharge over or through a "thin plate" was adopted for the sake of uniformity in practice—a definition of a member that could be easily stated, and the member could then be readily reproduced. Apart from this, however, it has no special value.

Suppose, instead, that we were to make a weir crest—to be adopted as the standard weir crest—semicircular in cross-section, and of some adopted standard radius of generation, say, 2.5 cm., if one likes—near enough to 1 in. to be called an inch, in the present instance; again, if one pleases, tangentially joining a solid curve of transition that shall fill the angle between the up-stream face of the weir and the bottom of the flume, somewhat following the line marked "Limit of Angle of Pressure" in Fig. 26, and, of course, to be defined in terms, to become normal (a catenary or a parabola); or a straight line, on a 2:1 slope, because it is readily constructed of wood. Would not the flow over such a weir be smoother, cleaner, less excruciating to behold or imagine? Would it not give more concordant results, and a smaller range of coefficients?

Next, let us consider the matter of observing the head on the weir. If the discharge were out of a tank, or reservoir, there would be, com-

monly speaking, no velocity of approach; and, to observe the height of still water up stream from the weir, and let it rule in the formula for discharge, would be a simple matter. This was the original condition of things, when weir experiments and experiments on the discharge of orifices were first made; when the apparatus for such experiments were mere playthings; but a weir without end contractions, near the down-stream end of a 6½ or 8-ft. flume, is quite another affair. To observe the depth on the weir in such cases, as has been generally done hitherto, is a mixing up of the study of hydrostatics and hydraulics, which long ago called forth the protest of the writer. Mr. Herschel.

This was done by an article* entitled "The New Hydraulics" ("The New Hydrometry", would have been a better title), some extracts from which may now be reproduced in this discussion:

"To the graduate student or practitioner in hydraulics [hydrometry] of the present day, there has always been something disappointing in the art. Here is a science which pretends to give rules and formulæ for the discharge of water through orifices and over weirs, when in truth it only indicates what these rules should be, and are not. Instead of being a science founded on fact, and deduced theory, as it set out to be, it has degenerated into a prolix schedule of coefficients to be applied in hydraulic cases as they are met with. So numerous have these coefficients become, that their great number has defeated their very purpose. It has been found impossible to match cases as they arise, with their appurtenant coefficients, and engineers have been obliged to practically ignore the most of them, and, instead of using them, to confine themselves in their practice to a very few selected simple cases. Instead of practicing under the guidance of a science of hydraulics, the working hydraulician takes for his mentors only a few simple hydraulic experiments, and by repeating with slavish exactness the apparatus used in the experiments to be followed, he manages merely to allow nature to do for him what she did years before for another. Any step, therefore, that will do away with or merely will reduce the range of the multitude of coefficients now found recorded in hydraulic writings for the practice of hydraulics, will benefit that science very much.

"Hydraulics, to look at the situation from another point of view, has not hitherto been able to free itself from the apron-strings of its older sister science of hydrostatics. Instead of noting the conditions obtaining strictly for the water in motion which is to be observed, and confining himself to them alone, the modern hydraulician still encumbers his work with measurements based on, or derived from, a water level which belongs in the domain of the hydrostatics of the case in hand, rather than in that of its hydraulics. Such cases are the measurement of the head on a weir, by measuring from the level of still [actually: running] water upstream from the weir; and determining the head on an orifice, from the position of the orifice and the level of still water upstream from it. Inasmuch as this mode of procedure totally eliminates the form and shape of the weir or orifice

* *Engineering News*, November 10th, 1898.

Mr. Herschel. —takes no account of the same whatever—the measurements thus taken being precisely the same for any and all kinds of weirs and orifices; and inasmuch as these different kinds of orifices do have, nevertheless, very different unit discharges, it becomes necessary to introduce a host of coefficients to cause this method of procedure to produce the numerous called for practical results.

"But how would it be, were we in these cases to take measurements and observations which would themselves be influenced by the shapes of the orifices and by the velocity of the water passing through them? May we not hope, that the coefficients to be applied to formulæ containing data thus obtained, would have a much smaller range and would be much reduced in number, to the manifest benefit of the art and science now spoken of? * * *

"A method of doing hydraulic work in the manner alluded to must be the next great step to take in the development of practical hydraulics, and there are many signs of the times, plain enough to those who will see them, that point in the stated direction."

Bazin has left us a significant proposition in this line. On page 238 of his final essay in the *Annales des Ponts et Chaussées,* 1898, the last section of the essay and at the very end of the account of all his weir experiments, which occupied his time for 50 years,* he says:

"*Pressure Under the Sheet of Water (Nappe) and Distribution of Velocity in Its Interior.*—The pressure under the nappe was measured in nearly all the experiments; as also, for a certain number of them, the distribution of pressures and velocities in their interior. This distribution has a theoretical value; but not so in the case of the pressure under the nappe, which in certain cases may be utilized. Given a weir of a certain type, its coefficient, m, will depend principally on two elements, namely: the contraction at the crest, and the pressure under the nappe; this pressure being so closely dependent on the discharge, that, by observing it continuously, one could deduce from it the variations of discharge with a precision which cannot be obtained by measuring the head on the weir directly.* * *"

Bazin then goes on to show, mathematically, that the method proposed is applicable to weirs of all kinds and forms of construction:

"The measure of pressure can thus be utilized to register variable discharges. An instrument founded on that manner of procedure has been in use in America for some years to meter the discharge of large conduits. It is well known, following some experiments of Venturi, that if one constricts on a short length of conduit, its diameter, a diminution of pressure is produced, corresponding to the produced increase of velocity. Determining now, by suitable rating, the relation that exists between these two elements, it is plain that a measure of pressures will lead to a measure of discharges, and that by automatic

* Bazin's experiments on weirs are only imperfectly known in the United States. As usually quoted, they are referred to as found in the translation by Marichal and Trautwine of his article in *Annales des Ponts et Chaussées,* of 1888, II. The complete list of these articles is as follows: 1888, II; 1890, I; 1891, II; 1894, I; 1896, II; 1898, II.

registration, one obtains the discharge of the conduit during a stated time, in spite of variations of flow. On this principle is founded the apparatus devised by Mr. Clemens Herschel.* The same principle is evidently applicable to weirs, and one could even cause them to register their discharge automatically, by using the apparatus described by Engineer Hegly to translate a record of head into a record of discharge. A measure of pressures under the nappe, showing with greater sensitiveness than a measure of head, all the variations in the régime of the falling nappe, may thus render useful service in certain especial cases where great precision may be necessary." Mr. Herschel.

To resume quotations from the writer's article, previously mentioned:

"By taking heights on the weir, as measured by means of a pressure tube laid in the body of the crest of the weir [see page 217 of Bazin's article, 1898] and basing the formulæ for the discharge of weirs on such hydraulic heights, instead of on heights derived from still water, which may be called hydrostatic heights, he [Bazin] gets again, as in the experiments above recited, a much smaller range of coefficients for his formulæ. M. l'inspecteur general Bazin notes with pleasure that this mode of observation and record gives more accurate results than the hydrostatic method, and is to be followed by preference.

"Nothing is wanting, then, to enable engineers to use the method of hydraulic coefficients exclusively, except an advocacy and teaching of the practice and a collection of the new coefficients, derived from experiment."

No such work has been done, however, in the 20 years that have passed since these words were written. To some this may be a source of discouragement, or even a reflection on the cogency or validity of the suggestion made. Let one consider, however, that change and invention, in despite of all boasting on the subject, move but slowly. Engineers are fettered by the forces of inertia quite as much as other people. Old-time habits, the teachings of school books, the constraining influences of ruts and channels of thought, control them only too rigidly, or else we should long ago have seen an improvement made in the practice of hydrometry, such as has been here advocated.

Bazin, after spending the better part of a lifetime on hydraulic experimenting, leaves such advocacy as his last words to the Profession. The writer did not hesitate, 20 years ago, to argue to the same end, and 20 years of further experience has only served to confirm him in those views.

Passing now to proof that the hydraulic system of pressure observations in hydrometry, and smooth running water, lead to a small range of coefficients in the appurtenant formulas, let us consider an instrument for metering water, other liquids, and gases, that makes use of those elements of hydrometry, referring, as many readers will have at once recognized, to the Venturi meter.

* *Transactions*, Am. Soc. C. E., Vol. XVIII (1888).

Mr. Herschel.

As an advisory member of the Power Test Committee of the American Society of Mechanical Engineers, the writer has recently had the duty imposed on him of compiling the coefficients that have been found by experimenters during the some 30 years' life of the Venturi meter. These coefficients are presented in Table 5.

Explanation of Table 5.—Column 1 gives the size of the end diameters and of the throat of the meter tube; also, the nature of the test or experiment, whether volumetric (in a tank) or by comparison with a weir, and the authority for the statements made.

Column 2 refers, respectively, to the coefficients to be applied to the theoretical discharge of the Venturi tube, or of the complete Venturi meter, to attain the discharge with which it has been compared, whether tank measurement or weir. In the latter case, it does not follow that either the coefficient given, or the weir discharge from which this coefficient has been derived, is correct.

There is also given, when it has been observed, the loss of head, in feet, of water column, end to end of the meter tube.

Both coefficients and losses of head are entered as appurtenant to the velocities of water through the throat of the meter tube, in feet per second, as given in the top main horizontal row.

Columns 3 to 9 give, alternately, the coefficients and losses of head, just mentioned, according as they have been found for the complete Venturi meter, or for the meter tube only, the latter requiring an observer, notebook, and pencil, and computation or diagram, to result in knowledge of the quantity of water discharged.

Column 10 states the construction of the meter tube. It is to be understood, however, that the throat of the meter and a short section at the up-stream end are invariably made of, or lined with, bronze.

Comment appears unnecessary. Attention, however, should be called to one other feature of the general art of making measurements, revealed by the experience at Lancaster, Pa., with the Venturi meter, with tank tests, made by F. H. Shaw, M. Am. Soc. C. E., during a period of seven years. Though the average of 44 such tests differs from unity by only $\frac{1}{28}$ of 1%, yet the two extreme variations were + 1.10 and — 1.89 per cent.

Why should there be any variations as wide as this from the true average?

To ask this question, is to answer it. There is probably no simpler operation in engineering than the measurement of the length of a base line. Yet every engineer knows that, were he to measure it a thousand times, no two of the measurements would be precisely the same, except by chance, even when made by the same implements;

Mr. Herschel.

TABLE 5.—VENTURI METER COEFFICIENTS.

American Society of Mechanical Engineers, Power Test Committee.

(1) Velocity through the throat, in feet per second.		(2) 5		(3) 10		(4) 20		(5) 30		(6) 40		(7) 50		(8) 60		(9) Average.		(10) Remarks.
		Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	Meter.	Tube.	
1 by 0.33 ft., Tank test, Herschel	Coefficient	....	(0.940)	.....	0.987	....	0.995	....	0.999	....	(1.000)	....	.....	....	.....	.....	.....	Made of hooped wood
	Loss of head	....	0.06	.....	0.26	....	0.97	....	2.10	....	.....	....	.....	....	.....	.....	.....	
1 by 0.5 ft., Tank test, Cornell Univ.	Coefficient	....	0.984	.....	0.987	....	0.991	....	0.987	....	0.983	....	.....	....	.....	.....	0.987	Cast iron.
	Loss of head	....	.....	.....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	
1 by 0.43 ft., Tank test, Cornell Univ.	Coefficient	....	0.988	.....	0.990	....	0.997	....	1.000	....	0.996	....	.....	....	.....	.....	0.993	Cast iron.
	Loss of head	....	0.06	.....	0.25	....	0.95	....	2.10	....	(3.30)	....	.....	....	.....	.....	.....	
1 by 0.33 ft., Tank test, Cornell Univ.	Coefficient	....	0.989	.....	0.985	....	0.987	....	0.990	....	0.993	....	0.989	....	1.000	....	0.988	Cast iron.
	Loss of head	....	.....	.....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	
2 by 1 ft., Weir Compn., Holyoke Water Power	Coefficient	0.99	0.99	0.99	0.99	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	Cast iron.
	Loss of head	....	.....	.....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	
2.5 by 1.08 ft., Tank test, F. H. Shaw	Coefficient	....	.....	1.00	.....	1.00 The average of 44 tests during a period								....	.....	.....	.....	Cast iron.
	Loss of head	....	.....	.....	.....	of 7 years; extremes were +1.10 and −1.89%.								....	.....	.....	.....	
4 by 1.33 ft., Weir Compn., Herschel	Coefficient	....	1.00	.....	0.995	....	0.986	....	(0.978)	....	.....	....	.....	....	.....	.....	.....	Wood; inside riveted plate.
	Loss of head	....	0.03	.....	0.15	....	0.62	....	(1.44)	....	.....	....	.....	....	.....	.....	.....	
8 by 4 ft., Tank test, Pittsburgh Water Works	Coefficient	....	.....	1.00	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	Riveted plate.
	Loss of head	....	.....	.....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.....	.....	
9 by 3 ft., Weir Compn., Herschel	Coefficient	....	(0.994)	.....	0.986	....	0.978	....	0.968	....	.....	....	.....	....	.....	.....	.....	Wood; inside riveted plate.
	Loss of head	....	0.04	.....	0.18	....	0.77	....	1.54	....	.....	....	.....	....	.....	.....	.....	

Mr. Herschel.

and occasional measurements would vary materially from the true average.

The reason lies in the imperfections of human nature and abilities, in the "personal equation" and such similar defects, and not in the apparatus or method used.

Arrived at the close of this discussion, we can sum up our conclusions, by sketching how weir and orifice measurements may be made more accurate in practice and with a smaller range of discharge coefficients:

(1). Adopt, if possible by international agreement, a standard cross-section of the orifice edges, or weir crest other than a thin plate. This may be a semicircle, as has been suggested, of 2.5-cm. (1-in.) radius, being the outside surface of a 2-in. pipe, or pipe of some such diameter, in which may be fashioned the piezometer holes to transmit pressure.*

(2). Make the standard weir with a gentle channel of approach, up stream from and below the weir crest, as advocated in the beginning of this discussion.

(3). Make a series of experiments on the discharge through such orifices, and over such weirs. With the number of hydraulic laboratories happily already in being, this ought not to be a difficult matter, especially, if this experimental work were apportioned among them.

(4). Set up formulas and tables of coefficients to represent the results of these 20th Century hydraulic experiments.

Mr. Molitor.

DAVID A. MOLITOR,† M. AM. SOC. C. E. (by letter).—This paper has been read with much interest, and, as a full account of all details and observations is given, it is possible to use the results for the verification of other formulas, aside from the one chosen by the author.

It might be expected that a purely empiric formula, like that of Bazin, based on heads up to 1 ft., would still give approximately correct results up to heads of 2 ft., and the author claims to substantiate this formula up to heads of 4 ft. However, this would, at best, hold only for absolutely identical conditions and weir dimensions.

In the Cornell experiments, made in June, 1899, by the late G. W. Rafter, M. Am. Soc. C. E., with a flume 6.53 ft. wide, and a standard sharp-crested weir, 5.26 ft. high, with a range of heads from 0.7 to 6 ft. on the crest of the weir, the agreement was not as satisfactory, showing clearly the practical limitations of an empiric formula as soon as any of the principal dimensions are changed. These experi-

* Bazin used a small tube, of about 0.6-in. outer, and 0.2-in. inner, diameter, to receive the pressure from under the nappe and convey it to the measuring instrument; but this tube was laid in the body of the weir and did not constitute the crest of the weir, as is now proposed.

† Detroit, Mich.

ments* offer an exceedingly interesting comparison with the author's results. Mr. Molitor.

The Bazin formula, given by the author, contains three factors, the first of which represents the empiric coefficient generally designated by $\frac{2}{3}\,\mu$; the second factor is of irrational form, and the third represents the discharge through a lateral orifice. It is patterned after Weisbach's formula, proposed about 1844, as follows:

$$Q = \tfrac{2}{3}\,\mu\left[1.041 + 0.3693\left(\frac{H}{p+H}\right)^2\right] L\,H\,\sqrt{2\,g\,H}$$

The writer has given a general discussion† showing the incorrectness of the several formulas quoted by the author. The King formula was not previously known to the writer, but as it is theoretically the same as the Bazin formula, with slightly altered numerical constants and the exponent of H changed from $\frac{3}{2}$ to 1.47, the same objections apply, even though the apparent agreement with the author's experimental results is better than for the other formulas as shown in Table 4.

Any formula of general applicability must express the true theoretic influence of all the hydrostatic and hydrodynamic pressures and weir dimensions on the discharge. The necessity for an empiric coefficient, μ, is merely to rectify the theoretic discharge to cover the unknowable effects due to adhesion, cohesion, friction, and contraction. Such coefficients have a forced application when they are made to cover up inherent defects in a formula merely for the purpose of agreement with certain experimental data of limited scope. The mere fact that hydraulic experiments are generally confined to reasonably small cases, makes it the more necessary that the theoretic form of the formula adopted shall be rational, so as to afford some assurance of reliable results when applied to extensive weir problems such as are frequently encountered.

A keen appreciation of this circumstance prompted the writer in 1908 to publish the formulas proposed by the late Gustav Ritter von Wex.‡ These formulas served a most valuable purpose in solving the intricate problem of flow over the triple flight of locks at Gatun, on the Panama Canal, in order to supply the necessary hydraulic data for the design of the emergency dams with which the writer was intrusted during 1907-08.§

It may be of interest, therefore, in connection with this paper, to show the nature of the empiric coefficient, $\frac{2}{3}\,\mu$, as found by substituting the author's experimental data in the more rational Wex formula.

* Given on p. 82 of the writer's "Hydraulics of Rivers, Weirs and Sluices."

† Appendix A of "Hydraulics of Rivers, Weirs and Sluices."

‡ "Hydraulics of Rivers, Weirs and Sluices."

§ The solution of this problem is given in Appendix B of "Hydraulics of Rivers, Weirs and Sluices."

Mr. Molitor.

Using the author's nomenclature, Formula 22, for a uniform approach channel and vertical sharp-crested weir, becomes:

$$Q = \tfrac{2}{3}\,\mu\, L \sqrt{2\,g}\left(\frac{H}{H+\frac{p\,v^2}{g\,H}}\right)\left[\left(\frac{v^2}{2\,g}+H+\frac{p\,v^2}{g\,H}\right)^{\frac{3}{2}} - \left(\frac{v^2}{2\,g}\right)^{\frac{3}{2}}\right] \ldots (A)$$

$$\text{Where } v = \frac{Q}{L\,(p+H)}$$

$$\text{and } \tfrac{2}{3}\,\mu = 0.3818 + 0.05\,\frac{H}{p+H} + \frac{0.16}{H} + 0.00013\,L \ldots\ldots (B)$$

Taking the author's experimental values of H and Q, as given in Table 4, also $L = 6.558$ ft. and $p = 3.72$ ft., the values of $\frac{2}{3}\,\mu$ were computed by Equation (A) and also by Equation (B), giving the results shown in Table 6.

The values of Q are 6.558 times those given in Table 4, and represent the total discharge, instead of the "discharge per foot of weir crest." It is manifestly incorrect to compute coefficients of contraction for a flow per foot of crest, for reasons clearly seen from Fig. 11, showing the distribution of velocity over the flume section and the resulting contraction adjacent to the wetted perimeter. In other words, the discharge over a weir of length, $2L$, is not equal to $2Q$, but is larger, and this difference is taken care of by the particular value of $\frac{2}{3}\,\mu$, and not by L. Where there is any appreciable end contraction, this effect becomes still more important.

TABLE 6.

H, in feet.	Q, in cubic feet per second.	$\frac{2}{3}\,\mu$ Equation (A)	$\frac{2}{3}\,\mu$ Equation (B)	Percentage of error.
0.4	5.797	0.4272	0.4275	− 0.07
0.6	10.486	0.4175	0.4162	+ 0.31
0.8	16.034	0.4117	0.4115	+ 0.05
1.0	22.402	0.4092	0.4093	− 0.03
1.2	29.550	0.4076	0.4082	− 0.14
1.4	37.440	0.4075	0.4077	− 0.05
1.7	50.556	0.4072	0.4078	− 0.14
2.0	65.108	0.4076	0.4082	− 0.14
3.0	123.330	0.4100	0.4103	− 0.07
4.0	195.645	0.4135	0.4126	+ 0.21
	Mean =	0.4119	0.4119	.....

Comparing the values of $\frac{2}{3}\,\mu$, as obtained from Equations (A) and (B), given in Table 6, it will be seen that the percentage of error is smaller in every case except the last one, when compared with the errors given by the King formula in Table 4, and the King formula fits the experiments more closely than any of the other formulas used for comparisons by the author. It should also be noted that the range in the

Mr. Molitor.

values of $\frac{2}{3}\ \mu$ for the different heads is quite small, giving a mean value of $\frac{2}{3}\ \mu = 0.4119$, while the range in the values of m in Fig. 24 is from 0.436 for $H = 0.4$ ft., to 0.4255 for $H = 0.8$ or 1.0 ft., and then increases to 0.465 for $H = 4.0$ ft.

It would seem pertinent to ask why any one should deliberately make a series of experiments to prove the correctness of a formula which is admittedly wrong in theory, and does not even fit the experiments on which it was originally based.

The experimental data from all sources certainly constitute the best scientific evidence, and when the collection of facts becomes sufficiently extensive to cover the entire range of practical application, there will be no great difficulty in establishing a rational weir formula.

The difficulty surrounding this problem is a decided shortage of facts, and not formulas. Hence, the scientific answer is to increase the assortment of facts to cover most, if not all, of the variables involved, thus reducing the element of speculation to a minimum.

In overlooking the Rafter experiments, the author evidently failed to avail himself of the best information so far given to the Profession. It might be interesting, therefore, to mention that Mr. Rafter conducted his experiments at the request of the United States Deep Waterways Commission, because the experiments and formulas given by Bazin were not deemed sufficiently wide in scope to justify their adoption in the solution of the many important questions which this Commission was required to answer. The results obtained by Mr. Rafter fully justified the views of the Commission.

A careful inspection of Fig. 24 shows quite conclusively that all experiments made on heads up to 1 ft. would lead to very erroneous conclusions when applied to heads much larger than this, and hence the high-head flows are those which must be investigated, if any valuable contribution to our knowledge on this subject is expected. As the hydro-chemical method of gauging is likely to fail on account of excessive cost and other technical difficulties regarding imperfect mixtures, etc., we must ultimately look to current-meter measurements for a final answer to large weir problems.

There can be no doubt of the high relative accuracy attainable by hydro-chemical gaugings, but whether the same degree in absolute accuracy can be secured is still to be proved.

After a careful comparison of the author's results with those of Bazin and Rafter, it appears that the discharges within the scope of actual measurements are in every case smaller than those observed by the author. This seems to indicate the presence of constant or periodic errors affecting either the author's experiments, or else all the other measurements combined.

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Although the weighing method is no doubt inaccurate within certain limits, the errors are more likely to be of a compensating nature, and repeated measurements would tend to reduce their magnitude.

The chemical method would probably involve smaller accidental errors, but might be afflicted with more or less serious constant errors, which would not be revealed until the results are compared with those obtained from an entirely different process of gauging, such as that used by Bazin and Rafter.

For instance, the determination of chlorine by the silver nitrate titration, with chromate indicator, is permissible only in neutral solutions and in the absence of organic matter. The presence of alkalis, acids, or organic substances in the natural pond water, unless carefully allowed for, would lead to constant errors in the results. It is also possible that some of the salt solution introduced at the dosing station never reaches the sampling station and is lost by absorption in the flume walls, etc. Furthermore, the range in velocity variation on any cross-section, as shown by Fig. 11, where the head = 2.97 ft., is from 3.3 ft. per sec. down to 1 ft. next to the wetted perimeter, and may be even less than this, would affect the time of travel between the dosing and sampling stations (108.7 ft.) by from 33 to 108 sec. or more. As a matter of fact, some of the salt introduced during an experiment might not reach the weir or sampling station until long after the experiment was over. Then again, the location of the holes in the sampling pipes might not give the peripheral flow sufficient weight in the samples taken, and frictional losses in the sampling pipes might account for some retardation in the relative strength of samples. All these influences might easily account for some periodic errors, which would not necessarily be of the same sign over the entire range of the experiments (as there may be different constant errors of opposite signs) and thus account for the fundamental differences or apparently large errors disclosed between the author's gaugings and those of Bazin and Rafter.

All this would lead to the inference that it was a mistake to assume the hydro-chemical gaugings to represent the best attainable results simply because they apparently fit the Bazin formula, even though Bazin's own experiments do not agree so fully with his formula. As experiments are always more important than the formulas based thereon, the author might better seek for an explanation of the inherent differences between his own and other experimental data, and in the event that he succeeds in this, he will no doubt merit full credit for his ardent labors.

This discussion is offered with the sincere desire to assist the author in achieving such results as may be of the greatest possible value, both to himself and the Profession.

Mr. Stearns.

FREDERIC P. STEARNS,* PAST-PRESIDENT, AM. SOC. C. E.—Before discussing this paper in detail, it seems appropriate to review briefly the results of the principal weir experiments which have been made, showing, among other things, the great variation between the results obtained by different experimenters. These differences are shown graphically on the diagram (Fig. 25), which may assist to an understanding of the discussion.

In this review it is hardly necessary to go farther back than to the experiments† of the late James B. Francis, Past-President, Am. Soc. C. E., made in 1852, which introduced a new epoch in weir experiments. They were made on a large scale, with extreme accuracy, and covered a wide variety of conditions. He determined the discharge over weirs with and without end contractions, with heads on the weir ranging from 0.59 to 1.58 ft., and with varying heights of weir above the bottom of the channel.

These experiments were described in much detail, and a careful reading of the description furnishes strong evidence that everything possible was done to secure accurate results. The conditions under which they were performed were in most respects excellent. The close agreement of the deduced coefficients for each experiment with the general result of all experiments furnishes further confirmation of the accuracy of the work.

One of the first indications, based on careful tests, that a weir measurement might not be as accurate as was supposed, resulted from the experiments‡ made by Fteley and Stearns on a 5-ft. weir, in 1877. This weir in all its features was clearly well within the limitations set forth by Francis as necessary for the accurate application of his formula. The conditions for making these experiments were excellent, and most of the experiments gave results agreeing with the formula, representing them within 0.1 of 1%, but the discharges with heads on the weir corresponding to those in the experiments of Francis exceeded his by fully 0.5 of 1 per cent.

The next series of experiments was that of Fteley and Stearns, in 1879, on a 19-ft. weir, 6.55 ft. in height. These experiments were made for the purpose of calibrating the weir so that it might be used subsequently for determining the discharge through the Sudbury Aqueduct and for testing the accuracy of a current meter.

The weir was necessarily placed in a very short channel in a gate-house at the head of the aqueduct, the length of the channel from the point of a stone pier in the middle of the structure to the head wall of the aqueduct being only 8.25 ft. In order to furnish the required

* Mr. Stearns died on December 1st, 1919.

† "Lowell Hydraulic Experiments."

‡ "Description of Some Experiments on the Flow of Water Made During the Construction of Works for Conveying the Water of Sudbury River to Boston", by A. Fteley and F. P. Stearns, *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 1.

Mr. Stearns.

space below the weir for the discharge of the water, it was necessary to place the crest about 6 ft. from the end of the pier. On each side, an additional distance was available, but the lower screens were only 7.45 ft. up stream from the weir.

In view of these abnormal conditions, it was expected that the results obtained by this weir might vary considerably from those obtained

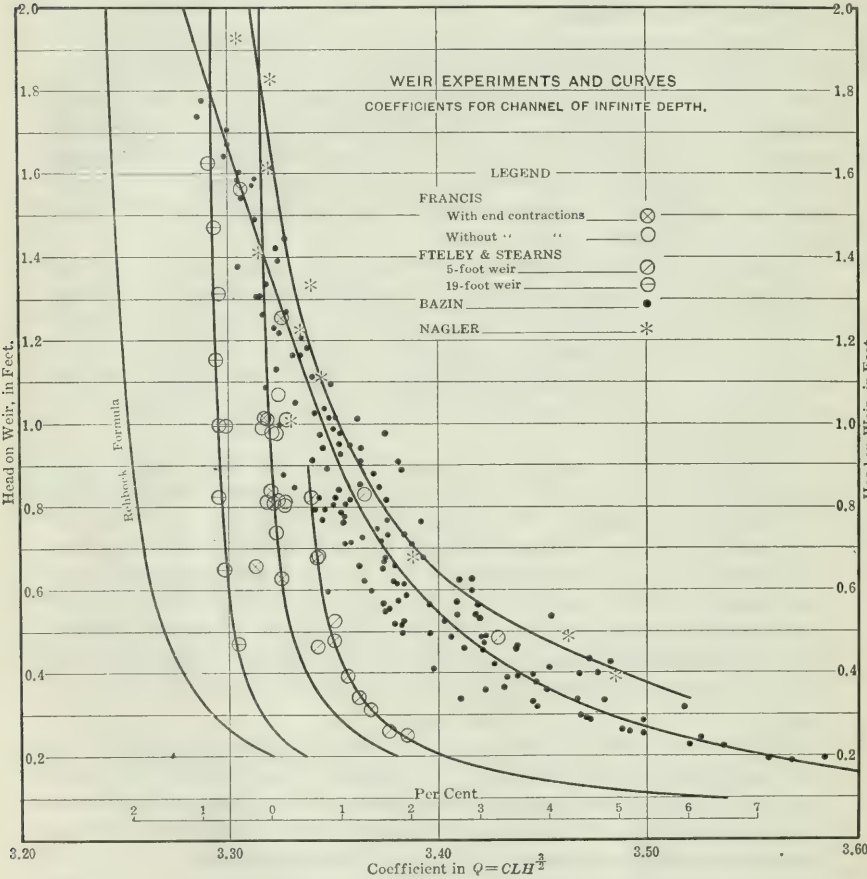

FIG. 25.

under normal conditions; but, with the smallest head covered by the experiments, the flow of water to the weir appeared to be so regular and the velocity of approach so small (0.151 ft. per sec.) that it was thought the results obtained with small heads might be normal.

The experiments produced a consistent series of coefficients for all heads on the weir, but they were about 0.7 of 1% smaller than

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those determined from the experiments of Francis, and, therefore, about 1.2% smaller than those determined from the 5-ft. weir. Aside from the abnormal conditions due to the short channel, the conditions under which these experiments were made were ideal.

The next set of experiments to cause additional doubt as to the accuracy of weir measurements was that of Bazin* on standard weirs having lengths of 6.56, 3.28, and 1.64 ft., respectively, and a height of weir above the bottom of the channel of 3.72 ft. for the first two and 3.30 ft. for the last.

These experiments were made under favorable conditions, in some respects, as the channel of approach was of good length, and there was a good measuring basin; in other respects, the conditions were unfavorable, as no provision appears to have been made for stopping oscillations or for regulating the velocities in the channel by using screens. Bazin states that, even in the stilling basin at the side of the channel, the oscillations were such that it was not feasible to determine the head on the weir by the hook-gauge alone, and he therefore used a dial operated by a float for this purpose, making comparisons, however, during an experiment, between the readings of the dial and of the hook-gauge, in order to get the proper relation to the crest of the weir.

As might be expected, under these conditions, the individual experiments do not follow closely the formula or curve representing the mean result, but the experiments are numerous, and their mean result with different heads on the weir is well indicated and consistent. This mean result agrees fairly well with the general result of the experiments of Francis when the head on the weir is about 1.5 ft., but exceeds his result by about 2% when the head is between 0.5 and 0.6 ft.

A much more recent set of experiments is that made by Th. Rehbock† in the hydraulic engineering laboratory of the Technical University in Karlsruhe. The speaker is indebted to Professor Lewis F. Moody, Consulting Engineer, for a translation of two short papers by Rehbock, describing the results of his experiments.

The experiments were carried on for several years, with an improvement in the testing outfit, from time to time, in the methods of experimenting, and in the accuracy of the observations, so that the paper of 1912 states that "the exactness of the results cannot be exceeded". In this paper, a formula is given for the discharge over standard sharp-edged weirs, with contractions suppressed.

The paper of 1913 states that it was decided to repeat the tests

* "Expériences Nouvelles sur l'Ecoulement on Déversoir", by H. Bazin, *Annales des Ponts et Chaussées*, 1888. Translation by Marichal and Trautwine, *Proceedings*, Engineers Club of Philadelphia, Vol. 7, pp. 259–310.

† *Zeitschrift* des Verbandes Deutscher Architekten- und Ingenieur-Vereine, 1912, Pt. 1; 1913, Pt. 1.

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in order to make sure that the formula of 1912 was absolutely exact, and, after making 191 additional tests, it was found that it was sufficiently correct for all practical cases except for heads on the weir less than 0.09 ft. By changing the formula slightly, it was found that it would apply correctly to smaller depths on the weir.

The experiments were on a small scale, the heads on the weir ranging from 0.03 to 0.59 ft., and the height of the crest of the weir above the bottom of the channel varying from 0.41 to 1.64 ft. The length of the standard weir was 1.64 ft., but tests were also made on a shorter weir. The individual experiments are not given; hence, in preparing the diagram (Fig. 25) only the line representing the formula could be shown.

Rehbock's results with heads on the weir of from 0.5 to 0.6 ft. are nearly 2% less than those of Francis. Hence, the inclusion of his results gives a range of experimental coefficients, obtained by different experimenters with these depths on the weir, of about 4 per cent.

Rehbock calls attention to weir experiments by Hansen, and shows their results by a diagram which indicates that Hansen's results with depths of from 0.5 to 0.6 ft. are 0.7 of 1% less than his, so that the range between Hansen and Bazin with these depths is about 4.7 per cent.

The next series of weir experiments is the one under discussion, and the precedent set by all other experimenters of obtaining results with depths on the weir of about 0.6 ft. beyond the limits reached by any previous experimenter remains unbroken. The author's results with this depth on the weir are about 2.5% greater than those of Francis, and consequently 4.5% greater than those of Rehbock, and more than 5% greater than those of Hansen. This is the present record for variation in weir coefficients with this depth on the weir, and it remains to be seen whether the next experimenter will be able to obtain results which will widen the range still farther.

In view of these large differences in the results of carefully conducted weir experiments, two conclusions seem to be warranted:

(1) That in the present state of the art the weir is not an accurate instrument for the measurement of water;
(2) That the art should be developed so that the causes of the variation in results may be discovered and eliminated.

It is much easier to ask the question, "What are the causes of these variations?" than to offer even a partial solution of the problem. The subject may be discussed profitably, even if the result is only to eliminate certain factors which are not the causes.

Notwithstanding the tendency which develops at times to accept a given weir formula and to ignore the results of others, on the ground

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that they are less trustworthy, it must be clear to those who have carefully examined the descriptions of experiments which have been made, that the variations, as a rule, are not the result of inaccuracy of physical measurements, except possibly with very small heads.

Clearly, it is not difficult to measure within a small fraction of 1% the length of a weir, the size of a measuring basin, or the interval of time required for the discharge from a weir to fill the basin if it is of sufficient size; and, as a rule, the basins have been of ample size, especially with moderate heads on the weir. The measurement of the head on the weir with a similar degree of accuracy is more difficult, but this may be accomplished, and has been accomplished even with small heads.

There is sometimes a small leakage from or into a measuring basin, but this is easily determined by observing the rate at which the water lowers before the experiment is made and after it is concluded, so that a correction practically free from error may be made.

It is customary not to empty a measuring basin completely, but to determine the volume between two water levels. In a long basin, oscillation from end to end, especially after the experiment is finished, cannot be wholly prevented, but the elevation of the water can be determined each minute for a sufficient length of time to cover one or several complete oscillations, and in this way get the correct level with minute accuracy.

As an illustration of the accuracy of such work under extreme conditions there may be cited the case of a measuring basin about 2 miles in length, where the distance between the two water levels was measured at three points with closely agreeing results. In measurements made on November 28th, 1879, the difference in level by three gauges was as follows:

Gauge	1	2.9632
"	2	2.9620
"	3	2.9582
Average		2.9611

The maximum difference from the average is 0.0029, or about 0.1 of 1%, and the average is probably still nearer the correct quantity.

An analysis of all the features of the more important weir experiments where the discharge into a basin has been measured shows that the discrepancies, except possibly those incidental to small heads, cannot be attributed to errors in physical measurements, and, therefore, their causes must be sought in the conditions under which the experiments were made.

It seems likely that the most important causes of the differences may be found in the differing motions of the water in the channel ap-

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proaching the weir, in the manner of taking the head on the weir, and in the oscillations occurring in the surface of the water in the weir channel.

In the future much more attention than has been given in the past should be devoted to the regulation of the relative velocities of the water in different parts of the cross-section of the channel of approach, as there is some reason to believe that this is an important factor affecting the discharging capacity.

The speaker made a study of the motions of the water in the weir channel in connection with the experiments of Fteley and Stearns on the effect of velocity of approach. Where there is a long channel of approach to a weir without end contractions, the water moves forward without being affected in any way by the presence of the weir until it reaches a point distant from the weir about $2\frac{1}{2}$ times its height above the bottom of the channel; then, at the bottom of the channel, the retarding effect of the weir begins to be felt to an increasing extent as the water approaches the weir. This has been shown by measuring with a current meter the velocity of the water at the bottom of the channel at various distances from the weir.

The retardation of the current at the bottom of the channel is accompanied by an increase of pressure, so that there is in the angle between the weir and the bottom of the channel, what may be called an angle of pressure, where the head is in excess of that corresponding to the height of the surface of the water in the channel above the point where the surface curvature begins.

Fig. 26 shows the approximate limits of the angle of pressure and the extent of the pressure within it, as determined at several points when the head on the weir was 0.8047 ft., and the velocity head due to the mean velocity in the channel of approach 0.0049 ft. The figures shown are decimals of a foot, those with the plus sign showing a pressure greater than the height of the surface of the water at a point 6 ft. up stream from the weir, and those with the minus sign a pressure smaller than that 6 ft. from the weir.

The line drawn at an angle of 45° from the crest of the weir to the bottom of the channel limits approximately the triangular area in which there are irregular motions, rather than a steady current toward the weir opening, a subject which will be discussed subsequently.

It is obvious that where the bottom water is retarded there must be a corresponding acceleration of the velocity of the water at higher levels, and as there appears to be no force to cause this acceleration except that of gravity, there must be a lowering of the water surface beginning at a point vertically over the origin of the angle of pressure. The lowering at this point, however, is such a minute quantity that it is not perceptible, and it is only somewhat farther down stream that the beginning of the surface curve can be detected.

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As the moving water approaches the weir, it necessarily mounts upward to pass over the crest, and it is natural to think of this process as occurring rather uniformly throughout the whole width of the channel. It is quite likely that this would be the case if the velocities were uniform in all parts of the cross-section, but where there is a higher velocity in one part of the width of the channel than in other parts this uniformity no longer exists.

Unless precautions are taken to prevent such action, the velocity of the water in the middle of the channel is likely to be greater than that at the sides, and, consequently, a greater pressure is produced within the angle of pressure at the middle of the channel than elsewhere. This condition cannot exist without causing lateral currents within the angle of pressure, and these in turn cause whirling motions which appear to be accelerated by the discharge of the water at the weir opening.

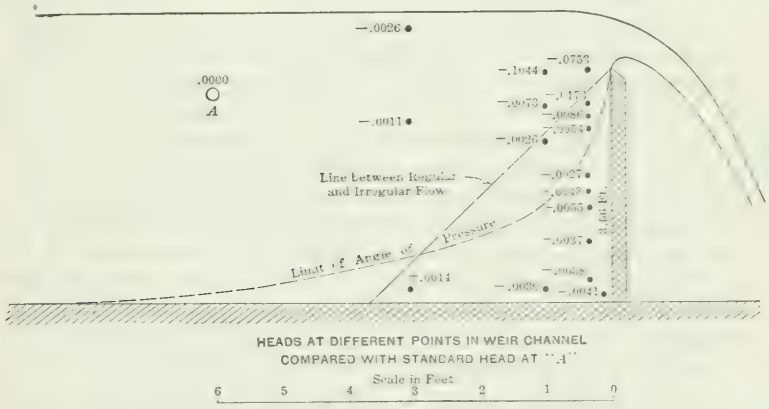

FIG. 26.

These movements may not only be deduced from theoretical considerations, but they have been observed practically. The speaker once had a small channel and weir constructed with one side of the channel of glass, for the purpose of studying these motions. The principal dimensions of the apparatus were as follows:

Width of channel and length of weir	1 ft.
Height of weir	1 "
Length of channel	8 "

A false bottom was provided by which the depth of the channel could be diminished to 0.50 ft. The motions of the water were made visible by mixing with it partly water-logged saw-dust.

There appeared to be an angle between the face of the weir and the bottom of the channel, extending up stream for about the height of

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the weir, in which there was no regular flow. The water frequently entered this angle near the middle of the channel, reached a point near the weir, next turned horizontally toward the side of the channel, then moved upward with an accelerating spiral motion, and passed over the weir. At times the motions within this angle were very irregular.

During the experiments on the 19-ft. weir, motions in the angle above the weir, such as were observed with the weir having the glass side, evidently took place. The velocities in the middle of the channel of approach were greater than in other parts of the channel, and it was found that partly water-logged saw-dust discharged through a pipe at the foot of the weir in the middle of the channel did not pass over the central part of the weir, but all of it appeared at the ends. There was evidently a strong spiral movement in this case, as it caused a swelling of the sheet at the ends, and adjoining the swelling there was a depression of nearly equal extent.

It seems to the speaker not improbable that these movements may be an important factor in causing the variation in the results obtained by different experimenters, and that in making experiments in the future and in the application of the results of experiments to practice, special care should be taken to produce a uniform velocity in all parts of the cross-section of the channel of approach, so that these irregular movements which have been described may be minimized.

The speaker once made experiments—which impressed him deeply —as to the possible effect of these upward spiral movements at the ends of weirs. In these experiments he observed the time required to empty a basin, having a circular horizontal cross-section, through an orifice in its bottom. The first experiment was made when the water in the basin was perfectly still. The plug was removed carefully so as not to set up any motion in the water other than that caused by the discharge through the orifice.

The next experiment was made in the same manner, except that the water, before the removal of the plug, had a slight rotary motion. In this case the time required to empty the basin was just twice as great as in the first experiment, showing a very great difference in the coefficient of discharge of the same orifice as the result of what appeared in the beginning of the second experiment to be a very slight motion of the water above the orifice.

There was no velocity of approach to the orifice at the beginning of either experiment, in the sense that velocity of approach is motion in the direction of the orifice, but the slight rotary motion at the beginning of the second experiment was soon increased by the discharge through the orifice into a violent whirling motion which had a radical effect on the discharge.

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This case seems somewhat analogous to that of the weir, except that, in one case, the motion is downward toward the orifice and in the other it is upward. The velocity of the water approaching the weir may be so small as to be practically negligible as an element directly affecting the discharge, and yet, if the velocity in one part of the channel is greater than in another, it may start spiral motions, to be augmented by the weir discharge, and thus become practically important.

The speaker believes that the view is prevalent that a very long channel of approach to a weir is a desirable feature, and that in such a case no screens would be required. This does not accord with the views here expressed. Water after flowing a long distance through a straight channel acquires very different velocities in different parts of the cross-section, the maximum velocities being in the middle of the channel and the lower velocities around the margins. Such uneven distribution of velocities, therefore, would set up secondary currents in the angle just above the weir, which are undesirable.

It is desirable, of course, to have a sufficient length of channel above a weir so that minor oscillations above the final screens may be prevented by a raft, and that screens may be provided to produce a uniform velocity throughout the cross-section at some point distant from the weir more than $2\frac{1}{2}$ times its height.

An additional distance to the screens of from 10 to 20 ft. is also advantageous, in order to eliminate the minor boils and whirls resulting from the passage of water through a screen, to allow bubbles to come to the surface, and to cause the water to become level from one side of the channel to the other.

Screens should be designed with care, in order to produce the desired results. The coarse screen for equalizing the velocity in all parts of the channel should operate under a sufficient head so that the flow through its different orifices will not be materially affected by irregularities in the velocity of the approaching water. If a screen is made with vertical slats and there is considerable head, the drop of the water at the screen will cause the maximum velocity to be at the bottom of the channel. This should be guarded against, and there are many ways in which it may be done.

A free fall of the surface of the water at a screen should be avoided, in order to prevent the formation of bubbles, and for other reasons. The final screen should be fine, and preferably with diverging openings and little fall.

The place of taking the head needs consideration, the main requirement, when there is a moderate velocity of approach, being that it be taken at a place where it will not be within the angle of pressure. As a rule, there would be no harm, provided the head is measured well above the bottom, in taking it somewhat nearer the weir than the

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upper end of this angle, say, at a distance from the weir not less than twice its height, but 6 ft. may be assumed as the minimum.

The method of taking the head is much more important. It is very rarely the case that a satisfactory measurement of the head can be made by direct measurement to the surface of the water above the weir, although the elevation of this surface is what is desired. The extended and accurate experiments of Hiram F. Mills, Hon. M. Am. Soc. C. E., on piezometers* show that "with currents flowing parallel with the side of a straight conduit, where the velocity is neither increasing nor diminishing, with orifices having edges in the plane of the side, and with passages normal thereto, the piezometric column will indicate the true height of the surface of the water in the conduit when in motion as well as when at rest." He calls attention to the necessity of attending carefully to these conditions when it is desired to obtain exact results.

The connection from the piezometric opening to the pail where the hook-gauge is placed may be made with a pipe, and, if there are no oscillations in the channel and the pipe is free from air, the water in the hook-gauge pail will stand at the same level as the surface of the water in the channel.

The speaker has tested a piezometer conforming to the requirements thus set forth, and used in connection with experiments on velocity of approach, by comparing the heads obtained with it with those obtained by direct measurement to the surface of the water, and the differences were only from 0 to 0.0003 ft. with low velocities of approach, and from 0.0003 to 0.0004 ft. with high velocities of approach.

If the water in the channel oscillates, the mean height of the water in the hook-gauge pail may be more or less than the average height of that in the channel, unless the connecting pipe and the orifices at both its ends have equal discharging capacity in both directions under a given head. For instance, if the entrance to the pipe from the hook-gauge pail were rounded, and that from the weir channel were square, and the pipe itself had equal resistance to the flow in either direction, the mean level of the water in the hook-gauge pail, as the result of oscillations, would stand lower than the mean level of the water in the channel.

It is probable that, even if the entrances to the pipe from the channel and from the hook-gauge pail are made identical, the mean level of the water in the latter may be different from the mean level of the water in the channel, by reason of the fact that the entrance to the pipe from the pail would be from quiet water, and that from

* "Experiments upon Piezometers used in Hydraulic Investigations," *Proceedings*, Am. Academy of Arts and Sciences, 1878, p. 52.

the channel would be from water flowing at right angles to the plane of the orifice. Mr. Stearns.

For the reasons stated, there can be no certainty that a correct measurement of the head will be obtained when there are oscillations in the channel.

Still one other feature needs to be considered: Mills' experiments proved that accurate measurements of the height of water in a channel may be obtained by using piezometers which comply accurately with his requirements when the water is flowing at a constant elevation, so that the pipe leading from the orifice to the gauging basin transmits pressure and does not cause a current through the pipe. If there are oscillations, there is necessarily a discharge into or from the orifice, and there is no evidence that, under these circumstances, the piezometric measurement will be correct. It is not difficult to prevent oscillations or to reduce them to extremely small amounts, except where they are caused by the wind, and the effects of the wind can be avoided by housing the apparatus.

When gaugings are to be made by the hydro-chemical method, it may not be feasible to suppress oscillations without at the same time suppressing the agitation and turbulence that are necessary for the thorough mixing of the salt solution with the water in the channel.

In order that, in the future, results may be produced which are consistent with one another and capable of reproduction at other weirs, the speaker submits the following as some of the conditions which should be observed:

1.—The weir should be one with end contractions suppressed. This is the general practice at the present time.

2.—A basin should be provided for measuring the water discharged by the weir, as this is the simplest and most accurate method of determining the volume.

3.—The weir channel should have sufficient depth, so that the velocity of approach will at all times be small.

4.—The weir and channel should be covered, and protected from disturbance by the wind.

5.—Arrangements should be made for a uniform flow, free from oscillations.

6.—Minor oscillations should be prevented by a raft above the screens.

7.—Care should be taken to produce an equal velocity in all parts of the weir channel. Current-meter measurements of the velocities in different parts of the channel should be made and recorded, and the screens should be modified until uniformity is attained.

8.—Piezometers should be provided on both sides of the channel, connected with separate hook gauges. These piezometers

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should conform with the requirements set forth by Mr. Mills. In the case of very wide channels, one or more additional piezometers may be required in the central portion of the channel.

9.—It is desirable that observations be made of the head in the angle at the foot of the weir, to determine whether the excess of head in this angle is normal; and other tests should be made in order to determine the movements of water in the angle up stream from the weir.

Coming now to a more direct discussion of Mr. Nagler's paper, the speaker appreciates the high degree of skill which has been displayed by the author in obtaining such consistent results in weir measurements by the hydro-chemical method. It is surprising that such uniform results could be obtained where so many different volumetric and chemical determinations were necessary.

Looking at the subject broadly, the speaker believes that, where the most accurate results obtainable are desired in the measurement of water over weirs at the present time, it is best to use a formula based on the results obtained by various experimenters rather than one deduced from any one set of experiments; and as this represents his idea of the most accurate results obtainable with present information, he obviously does not accept new results which for moderate heads on the weir are greater than those obtained by any other experimenter.

As has already been stated, the variation in the results obtained by different experimenters appears to be due to the conditions, known and unknown, under which the particular experiments were made, and, judging from the high discharge over the author's weir with moderate heads, the conditions in his case were especially unfavorable.

In order to compare the author's results with others, the speaker has recomputed them, after correcting the depths for the effect of velocity of approach, and in this way has obtained the corresponding coefficients for a weir of infinite height—or, in other words, for a weir in which the velocity of approach is negligible.

This correction has been made, not by applying Bazin's general formula for the discharge over weirs, which is quoted by the author, but by using the formula directly determined by Bazin from his experiments with weirs of different heights. According to this formula the observed head on the weir is corrected for velocity of approach by adding $1\frac{3}{3}$ times the head due to the mean velocity of the water in the weir channel. The coefficient, $1\frac{3}{3}$, is a generalization by Bazin of varying coefficients found with different heights of weirs and different heads on the weirs. Fteley and Stearns, as the result of

their experiments on velocity of approach, adopted 1.5 as a general coefficient in the same formula. Mr. Stearns.

After recomputing the results of the author's experiments in the manner indicated, so as to make them comparable with those obtained by others, it is found that with the smaller heads on the weir, ranging from 0.4 to 0.7 ft., his results are somewhat above those of Bazin, which, in turn, are considerably above those of all other experimenters. With a head on the weir of about 1.6 ft., the results agree well with those of Francis, Fteley and Stearns, and Bazin. With heads of about 4 ft. no comparisons can be made, because other experimenters have not dealt with such large heads, but the speaker believes that those experiments with very high heads have little general value, because of the high velocity of the water in the channel of approach.

The effect of velocity of approach on the discharge over weirs has been determined experimentally by causing the same quantity of water to flow over weirs of different heights and noting the effect on the head on the weir. In the extreme case in Bazin's experiments, the maximum effect was 0.144 ft. on a weir 0.80 ft. high; in the extreme case in the Fteley-Stearns experiments, 0.128 ft. on a weir 0.50 ft. high. Using the Bazin formula, the effect of velocity of approach in the author's experiment with maximum head would be 0.377 ft. on a weir 3.72 ft. high.

In most practical cases the difference in the effect of velocity of approach by the formulas of Bazin and of Fteley and Stearns is unimportant, but, as applied to the maximum experiment of the author, it amounts to a difference of 0.038 ft., which would affect the discharge by about 1½ per cent.

The author suggests that his experiments with large heads verify Bazin's formula for these greater heads, but, even if this were the case for a weir 3.72 ft. high, it would not verify the formula for other heights of weir, unless that part of the formula which takes account of the effect of the velocity of approach is strictly correct, and, because of the arguments above stated, there is no reason to think that any formula for velocity of approach can be depended on so far beyond the limits of actual experiment.

In regard to the coefficients in the weir formula for high heads, it may be said that experiments with orifices and nozzles show that the law that the discharge varies as the square root of the head is applicable to very high heads, and the same principle is involved in the weir formula when the heads are not small and the discharge is not affected by velocity of approach. Hence, one may reasonably, and probably without any serious error, use for a head of 4 ft. the coefficient deduced from experiments with a head of 1.5 ft. The uncertainty in practice arises more from the effect of velocity of approach, and it is the formula for making this correction that is

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especially important when dealing with high heads in cases where a great depth of channel cannot be obtained.

Attention is again called to Fig. 25, which shows the results obtained by different experimenters, after eliminating the effect of velocity of approach, so that the coefficients are applicable to a channel of infinite depth.

When Francis made his correction for velocity of approach, no trustworthy experiments had been made to determine the amount of such correction, and, consequently, he adopted a correction, based mainly on theoretical considerations, which gave satisfactory results with his experiments, in which the effect of velocity of approach was small.

Since that time, however, experiments by Fteley and Stearns and by Bazin show definitely that velocity of approach has greater effect than is indicated by the Francis formula. His experiments, therefore, have been recomputed, with larger corrections for velocity of approach than he used, and the diagram is based on the corrected results. The curve representing the mean result of his experiments as recomputed, was determined by the formula, $Q = 3.313\ L\ H^{\frac{3}{2}} + 0.006\ L$. This is a substitute for the Francis formula for suppressed weirs, which, for a channel of infinite depth, is $Q = 3.33\ L\ H^{\frac{3}{2}}$.

When the latter formula is used, the correction for velocity of approach should be that given by Francis, which is substantially an addition to the measured head on the weir of the head due to the mean velocity of the water in the channel of approach; but, if the revised formula is used, the addition to the measured head should be substantially 50% greater.

The curve representing the Fteley and Stearns experiments on the 5-ft. weir was computed from the formula, $Q = 3.33\ L\ H^{\frac{3}{2}} + 0.0065\ L$; that representing their experiments on the 19-ft. weir from the formula, $Q = 3.291\ L\ H^{\frac{3}{2}} + 0.004\ L$.

The Rehbock formula for a channel of infinite depth is

$$Q = \left(3.235 + \frac{0.0167}{H - 0.0094}\right) L\ H^{\frac{3}{2}}.$$

The curve representing Bazin's experiments is his experimental curve, and not one computed from his formula, which does not agree as well with his experiments.

The Nagler curve is one representing his experiments after they have been corrected for velocity of approach.

It is not the speaker's purpose in this discussion to pass on the merits of the different series of experiments, but it does not seem out of place to call attention to a prominent feature of the diagram,

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namely, that the experiments of Francis, Fteley and Stearns, and Rehbock, with the larger heads on the weir—say, those above 1 ft.—give coefficients which change very little with increasing heads. In this respect, they seem to be in harmony with the well-established principle, already referred to, that velocities vary as the square root of the head, except when the heads are small. With small heads and the consequent low velocities, such factors as the cohesion and viscosity of the water and its adhesion to the edge of the weir have a very much more important effect on the discharge than with large heads and high velocities.

Bazin's experiments, on the other hand, show rapidly decreasing coefficients as the heads increase, and, even with the larger heads on the weir, this marked decrease in the size of the coefficient continues.

Francis made a series of weir experiments at the Tremont turbine,* entirely separate from those on which his formula for the discharge over weirs was based, in which a constant volume of water was discharged over weirs of different lengths, and these experiments may be used to test the rate at which coefficients increase with decreasing heads on the weir.

These experiments were made with weirs having end contractions, and the conditions for maintaining an exactly constant volume of water over the weirs were not very favorable, as there was a somewhat variable head acting on the orifices through which the water was discharged; but the experiments were conducted with much care, and due account was taken of the variations in the head on the orifices.

By selecting only the experiments in which the discharge was least influenced by end contractions, results may be obtained which, though not conclusive, will indicate whether or not, within the range of the experiments, the experimental formula or curve tested gives coefficients increasing too much or too little with decreasing heads on the weir. Corrections for end contractions were made by using the rule subsequently adopted by Francis, namely, the reduction of the measured length of the weir by one-tenth of the head on it for each end contraction.

As the volume of water in these experiments was practically constant, the head on the shorter weir is obviously greater than that on the longer weir. If, therefore, one assumes the coefficient for the head on the shorter weir to be that given by any particular weir formula or curve, he may compute the corresponding coefficient for the smaller head on the longer weir. If this coefficient is larger than the coefficient for the smaller head, as deduced from the weir formula or curve under consideration, it furnishes an indication that the

* "Lowell Hydraulic Experiments," p. 88.

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coefficients in the formula or curve do not increase fast enough, and, if the coefficient is smaller, the indication is reversed.

The weir formulas or curves which may well be compared, because the experiments on which they are based conform most nearly to the range of heads in the Francis experiments at the Tremont turbine, are those of Fteley and Stearns for the 5-ft. weir, Bazin, and Rehbock.

By referring to Column 13 of Table 7 it will be seen that the differences of the experimental coefficient for the smaller heads from that by formula or curve are always plus in the case of the Fteley and Stearns formula for the 5-ft. weir, and practically the same statement may be made as to the differences in the case of the Rehbock formula. On the other hand, the differences in the case of the Bazin experimental curve are always minus. The average difference in the case of the Rehbock formula is + 0.34 of 1%, Fteley and Stearns formula, + 0.44 of 1%, and the Bazin experimental curve, — 1.11 per cent. In other words, the coefficients in the Rehbock and Fteley and Stearns formulas do not increase quite as rapidly with decreasing depths on the weir as is indicated by these experiments, and in the case of the Bazin experimental curve they increase more rapidly.

There are in this paper, and especially in the introductory portion, statements of a historical nature which are inaccurate. Thus, the author speaks of the experiments of Francis as covering a range in head on the weir of from 0.736 to 1.060 ft., whereas these experiments covered a range of from 0.592 to 1.569 ft.

In the next paragraph, referring to several experimenters, he says that they were unable to use higher heads, due to the fact that their volumetric measuring basins filled too rapidly with the large quantity of water which flowed over the weir at high heads. The Fteley and Stearns experiments were included in this statement. There was no such limitation in the experiments with the 19-ft. weir. With the maximum discharge, the basin filled in about 36 min.; it may be noted that the author discharged his volumetric tank containing the salt solution in about 15 min.

In the same paragraph he adds: "Nevertheless, these few experiments, within so small a range in head, were used by each experimenter in evaluating the coefficients for the standard weir formulas which bear their names." This statement is somewhat incorrect, because the Fteley and Stearns general weir formula, where it relates to heads of more than 0.5 ft., was based almost wholly on the experiments by Francis. However, the formula was given a form which would make it agree with these results of Francis and with those obtained by Fteley and Stearns with small heads on the weir, thus making a formula of more general application than that of Francis, which was not applicable to small heads on the weir.

TABLE 7.—WEIR FORMULAS AND CURVES, TESTED BY THE FRANCIS WEIR EXPERIMENTS MADE AT TREMONT TURBINE.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	FTELEY AND STEARNS FORMULA FOR 5-FOOT WEIR.											
	Shorter Weir.					Longer Weir.						Difference, experimental coefficient from that in Column 11. Per cent.
No. of series.	Head on weir, in feet.	Length of weir, in feet.	No. of end contractions.	Length, corrected for end contractions, in feet.	Coefficient by F. & S. formula.	Head on weir, in feet.	Length of weir, in feet.	No. of end contractions.	Length, corrected for end contractions, in feet.	Coefficient by F. & S. formula.	Coefficient from Francis' experiments.	
VII.....	1.1333	5.487	2	5.260	3.3354	0.5929	13.978	4	13.741	3.3442	3.3741	+ 0.89
VII.....	0.9553	6.987	2	6.796	3.3370	0.5184	16.980	4	16.773	3.3474	3.3823	+ 1.04
IV.....	0.6554	6.987	2	6.856	3.3423	0.3584	16.980	4	16.837	3.3603	3.3656	+ 0.16
III.	0.4715	6.989	4	6.800	3.3495	0.2917	13.978	4	13.861	3.3713	3.3769	+ 0.17
III.....	0.4681	6.987	2	6.893	3.3503	0.2552	16.980	4	16.878	3.3804	3.3991	+ 0.55
II.....	0.3745	6.987	2	6.912	3.3584	0.2339	13.978	4	13.884	3.3874	3.3873	0
VIII.....	0.4936	5.487	2	5.388	3.3488	0.2282	16.980	4	16.890	3.3896	3.3984	+ 0.26
Average..	0.6502					0.3541						+ 0.44

(1)	BAZIN EXPERIMENTAL CURVE.				REHBOCK FORMULA.			
	(Note: All columns omitted are the same as those above.)							
	(6)	(11)	(12)	(13)	(6)	(11)	(12)	(13)
No. of series.	Coefficient from Bazin curve.	Coefficient from Bazin curve.	Coefficient from Francis' experiments.	Difference, Column 12 from Column 11. Per cent.	Coefficient by Rehbock formula.	Coefficient by Rehbock formula.	Coefficient from Francis' experiments.	Difference, Column 12 from Column 11. Per cent.
VII.....	3.3354	3.3909	3.3741	— 0.50	3.2499	3.2636	3.2876	+ 0.73
VII.....	3.3488	3.4060	3.3942	— 0.35	3.2527	3.2678	3.2968	+ 0.89
IV.....	3.3804	3.4540	3.4039	— 1.45	3.2609	3.2829	3.2836	+ 0.02
III.....	3.4171	3.4860	3.4450	— 1.18	3.2711	3.2942	3.2978	+ 0.11
III.....	3.4185	3.5100	3.4682	— 1.19	3.2714	3.3030	3.3190	+ 0.48
II.....	3.4599	3.5265	3.4897	— 1.04	3.2807	3.3094	3.3089	0.02
VIII.....	3.4120	3.5350	3.4626	2.05	3.2695	3.3114	3.3179	+ 0.20
Average.....				— 1.11				+ 0.34

Mr. Stearns.

The author seems to suggest in his statement that a few experiments are not a sufficient basis for a formula, but this depends on the accuracy of such experiments. A reference to those on the 19-ft. weir by Fteley and Stearns, as represented by Fig. 25, shows clearly that the line representing them is practically as well determined by the number of experiments made as if they were multiplied many times.

He states, on page 154, that the points at which the gauges which he used tapped the flume duplicated the experimental points of observation used by Bazin, Fteley and Stearns, and Francis. Hence, his experiments seem to indicate that the discrepancies between the Bazin, Francis, and Fteley and Stearns formulas cannot be accounted for by the different points of observation used in their experiments.

In order to duplicate the Fteley and Stearns points of observation, he connected the stilling tank to the side of the channel 6 ft. from the weir, and 0.414 ft. below the weir crest. In the experiments on the 5-ft. weir of Fteley and Stearns, the head was taken about 1 ft. above the bottom of the channel, 6 ft. up stream from the weir, at a point in the channel 1.5 ft. from its side. In the experiments on the 19-ft. weir of Fteley and Stearns, the heads were taken through four double piezometers, each in the middle of a fourth part of the width of the channel, just below the level of the crest of the weir.

To represent the point of observation used by Francis in experiments on suppressed weirs, the author connected the stilling tank with the side of the flume at a point 6 ft. from the weir and 3.22 ft. below the crest. Francis took his observations in the middle of the channel at a point 6 ft. from the weir and 4.80 ft. below the crest. The difference in this case is important, because the head as taken by the author was probably outside the angle of increased pressure, and the head taken by Francis was within the angle of pressure, but not where the increased pressure was great. This was demonstrated in the case of the Francis experiments, because he found by test that the head at the point chosen, 6 ft. from the weir, was greater than the head 12 ft. from the weir. In the Francis experiments, however, the velocities were so low that the point of taking the head did not have much importance.

The author is rather optimistic about a number of features connected with the experiments. Thus, on page 154, in discussing the method of taking the head on the weir, he says: "Certainly the fact that the conducting pipe and tapping hole were 2 in. in diameter eliminated all error as to the nature of the tap." It would be rather interesting to know why the use of a pipe and tapping hole of a given diameter should eliminate error. He gives no information as to whether the taps complied with those features which Mills, from his experiments on piezometers, found necessary, namely, as to whether

the orifice was precisely in the plane of the side of the flume, and whether the pipe was precisely normal thereto. Mr. Stearns.

He says elsewhere in the paper that when the water in the flume had a velocity exceeding 3.3 ft. per sec. "the waves * * * were of so great a magnitude that the floating railroad ties were ineffective in decreasing the amplitude of their motions." With such oscillations, it would seem quite possible, for reasons already suggested, to record a head at the hook-gauges which was not the true head on the weir, notwithstanding what he refers to as "the remarkable agreement between the mean readings for all three hook-gauges and the piezometer tube."

The author speaks in one place of the excellent velocity distribution in most experiments, and, on page 125, presents velocity contours at a point 25 ft. above the weir, which show something far different from a uniform distribution, as the velocities range from less than 1 ft. per sec. near the surface to 3.2 ft. per sec. at mid-depth. The low velocity at the surface may have been due to the floating railroad ties a short distance above this section, and the conditions nearer the weir may have been more favorable.

On page 118, the statement is made that placing the weir at the end of the flume gave it a free basin about 90 ft. in length, which does not seem to agree with the statement that there were two floating railroad ties placed across the flume about 28 ft. above the weir.

It is rather interesting to note the optimistic conclusions of the experimenters who have obtained especially low and high results. Thus, Rehbock, who experimented with heads on the weir ranging from 0.03 to 0.59 ft., referring to the degree of accuracy of his formula, stated that he was convinced that the results to be obtained by it deviated not more than ½ of 1% from the true value.

The author, with results that are about 4.5% higher than those obtained by Rehbock, says: "There is no doubt that some experiments give the correct discharge within 0.1 of 1 per cent. In general, it might be said that the maximum variation from the true discharge is less than 0.6 of 1% for every experiment except No. 20."

B. F. Groat,* M. Am. Soc. C. E. (by letter).—Mr. Herschel gives some valuable suggestions as to weir coefficients. The writer adds that, perhaps, a correct application of the theory of similarity† may further simplify the question for geometrically similar weirs with proportional heads, provided the latter are not too small. Mr. Groat.

It should be observed, however, that to have exact similarity, under

* Pittsburgh, Pa.

† "Ice Diversion, Hydraulic Models, and Hydraulic Similarity", *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

Mr. Groat. given gravitational forces, the model weir should be tested with a different fluid from that discharged by the prototype. As an example, a weir about one-sixth of full size might be tested with mercury at a proper temperature. This calls attention to the fact that temperature observations have been omitted too frequently in the past.

In referring to the dynamic reactions of different fluids, it should be borne in mind that the same fluid under different viscosity conditions (as water at different temperatures) acts exactly as so many different fluids. Weirs under low heads are certainly very sensitive to changes in viscosity and, therefore, to changes of temperature.

Mr. Nagler. FLOYD A. NAGLER,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer had not intended to enter into a discussion comparing the results of all weir experiments with his own hydro-chemical gaugings. Engineering literature has worn this subject practically threadbare. In verifying the Bazin results, the hydro-chemical gaugings naturally differed from the conclusions of other experimenters; but the main intent of the paper was to offer a new set of experiments which, having been executed, was found to check with the formula for discharge of weirs, which Bazin deduced from his studies, far beyond the actual range in head of the Bazin experiments.

With the contribution to engineering practice of such valuable work as that performed by B. F. Groat, M. Am. Soc. C. E.,† the hydro-chemical method of measuring water becomes quite as reliable as any so-called "tank" or volumetric process when conducted with an equal degree of skill and care. In fact, the hydro-chemical method is a volumetric process and depends on the accurate measurement of quantities of solution; and the chemist can determine the number of parts of chlorine or other element in a given quantity of water with as great refinement and ease as the engineer can measure a base line. This hydro-chemical method, therefore, becomes compatible with any other volumetric process for measuring water in an effort to determine weir coefficients or check weir formulas. In neither case can exact values be obtained, but the truth is revealed in proportion to the accuracy, range, and square root of the number of measurements; and, thus, a sufficiently great number of experiments constitute an adequate basis for the "verification of any formula," or even the evolution of the formula itself.

The writer has mentioned that the experiments of the late J. B. Francis, Past-President, Am. Soc. C. E., were contained between the limits of 0.736 to 1.006-ft. head on the weir; he should have added that only the experiments on weirs with end contractions suppressed were referred to, being included in the series numbered 44-55 and 67-71,

* Albany, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXX (1917), p. 951.

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with a weir 9.992 ft. long. The writer did not attempt to make his work comparable with any experiments on contracted weirs. The mean depth of the channel behind this suppressed weir used by Mr. Francis was 5.048 ft. The pipes communicating with the hook-gauge chamber opened near the bottom of the channel, 6 ft. from the weir.* Mr. Stearns states that the writer's point of measurement for his Gauge 2 was not comparable with that used by Mr. Francis, although it was 6 ft. from the weir, because it was not about 4.80 ft. below the weir crest. However, since the writer's weir crest was only 3.72 ft. above the bottom of the channel of approach, he believed that he was fulfilling the conditions of the Francis weir by locating the point of measurement 6 ft. from the weir and near the bottom of the channel. Incidentally, Mr. Francis† found the difference in the head measured 6 ft. and 12 ft., respectively, from the weir to be "inconsiderable," and on an average equal to only 0.0014 ft. for heads on the weir very nearly equal to 1 ft.

The accuracy of the statement that Gauge 1 in the writer's experiments measured the head on the weir at a point comparable with that used by Messrs. Fteley and Stearns has also been questioned by Mr. Stearns. The point used by the writer was 6 ft. from the weir and 0.414 ft. below the weir crest, and, quoting from the paper by Messrs. Fteley and Stearns on "Description of Some Experiments on the Flow of Water Made During the Construction of Works for Conveying the Water of Sudbury River to Boston,"‡ we find that these experimenters used a point quite similar:

"The orifice in the glass was 0.04 ft. in diameter, and was originally 6 ft. from and 0.414 ft. below the level of the crest of the weir; it was afterwards moved 0.02 ft. higher."

This was the point used by Messrs. Fteley and Stearns in their experiments on velocity of approach, in which they varied the height of the weir above the bottom of the channel of approach; and although this point of measurement was not used in all their experiments, the writer adopted it as being representative, since it is also practically equivalent to the point of measurement used in their experiments on the 19-ft. weir.

In all the writer's gauges, the 2-in. pipe connecting with the hook-gauge wells entered the flume normal to the walls. The end of the pipe was carefully dressed with a file and was flush with the inside wall, in no case projected beyond the wood. The writer is of the opinion that when all other conditions are the same, the most accurate piezometer would be that which gives satisfactory results with a connection of maximum cross-section and opening with the body of

* "Lowell Hydraulic Experiments", p. 107, also Plate XIV, Figs. 9, 10, and 11.

† *Ibid*, p. 141.

‡ *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 9.

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flowing water. Such a piezometer would respond most readily and accurately to fluctuations in the flowing stream, give a result covering a greater average of static conditions, and be less likely to respond to the influence of small stray diagonal currents. A piezometer with a 2-in. orifice and short 2-in. pipe connection with the stilling well, should produce more accurate results than small $\frac{1}{4}$-in. or $\frac{3}{8}$-in. orifices with long rubber hose connections, similar to those used in the weir experiments of Messrs. Fteley and Stearns.

The writer still wishes to maintain his opinion that, in general, large scale weir experiments have been limited to those on low heads, due to the limitations in the size of the measuring basin. Operating under a head equal to the highest one on the writer's weir, the discharge over the Francis suppressed weir would have filled his basin in but 41 sec.; the Fteley and Stearns 19-ft. weir would have filled its enormous basin in but 9 min., and the standard Bazin weir would have filled its basin in less than $1\frac{1}{2}$ min.

Almost all weir formulas are practically identical in their introduction of the more significant variables such as L and H. The main difference is usually defined in their method of introducing the "velocity of approach" correction and its magnitude. Hence, in testing the applicability and accuracy of the different formulas, it would seem that the logical procedure would be to magnify the necessity for the introduction of this factor of velocity of approach, instead of minimizing it; and the formula which is the most accurate when a large velocity of approach correction becomes necessary, is that which most correctly incorporates this element. The writer, therefore, wishes to take exception to Mr. Stearns' statement that his high head experiments seem to "have little general value, because of the high velocity of the water in the channel of approach", and, for the same reason, he suggests by the title of his paper that his experiments verify the Bazin formula. Mr. Stearns also presents a comparison of the effect of velocity of approach in the extreme case for the Bazin experiments as 0.144 ft. on a weir 0.80 ft. high, and for the Fteley and Stearns experiments as 0.128 ft. on a weir 0.50 ft. high, whereas the writer's maximum case was 0.377 ft. on a weir 3.72 ft. high. Without mention of the measured head on the weir, however, this comparison is worthless and would lead one to infer that the greatest effect was that which occurred in the hydro-chemical experiments. This, however, is not the case. Expressed as a percentage of the discharge over the weir, the effect due to the velocity head of approach in each case is as follows: Bazin experiment, 17%; Fteley and Stearns experiment as much as 25%; and the hydro-chemical experiment, 17 per cent. Actually, therefore, Messrs. Fteley and Stearns, themselves, have presented the Profession with the experiment involving the maximum effect of the velocity in the

channel of approach to the weir. It also appears that the writer's experiments are within the limit of those of Bazin on the effect of velocity of approach, and in verifying the Bazin formula for high heads there is fair assurance of the applicability of this formula to weirs of other heights discharging water under similar heads. Mr. Nagler.

Far more important than the production of a uniform velocity in the channel of approach seems to be the production of smooth stream-line motion in the water approaching the weir. A channel with a mathematically uniform distribution of velocities is not in equilibrium, due to the increased friction near the sides and bottom. A channel of approach of considerable length is not only advisable but necessary to produce the best condition. The conditions may be improved by the addition of screens or baffles; but, in many instances, screens alone, with a short channel of approach, would not produce the desired condition. Messrs. Fteley and Stearns performed experiments in which the channel of approach was so short that it was necessary to place the screens as close as 12 and 7 ft. from the weir; in the latter case, the screens were only 1 ft. from the point of measurement of the head. Bazin intimates that this unfavorable condition may possibly be the cause of the constant differences in the two Fteley and Stearns series of experiments and the discrepancy between their results and those of other experimenters. It is interesting in this connection to note that the production of eddies or a non-uniform distribution of velocities near the weir would tend to decrease the discharge over the weir; whereas, those experimenters using the shorter channels of approach, necessitating the use of screens, secured the smaller discharge, while those using the longer channels of approach, with screens far distant from the weir, or none at all, secured the higher coefficients of discharge.

When the writer mentioned waves of magnitude in the channel of approach for the experiments with the greatest discharge, he did not wish to convey an exaggerated impression of the unfavorable condition. In the worst case, the waves were only 1 or 2 in. high, and, being momentary, did not affect the constancy of the hook-gauge readings and, therefore, were not considered objectionable.

The distribution of velocities shown in Fig. 11 was the result of current-meter gaugings taken early in the progress of the hydro-chemical experiments. This condition was improved immediately by shifting the maximum velocity from the left to the center of the stream. The low velocity at the surface was attributed to the floating raft, and disappeared with approach to the weir. The object of the raft was to decrease the surface vibrations. As the ties only sank 3 or 4 in. in a channel from 4 to 8 ft. deep, the writer considered them negligible when the statement was made that the basin was free for 90 ft. above the weir. Fig. 11 shows more than 70% of the section

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with a velocity greater than 3 ft. per sec. and not exceeding 3 ft. per sec. by more than 10 per cent. Comparatively, this is a far more uniform distribution of velocities than that observed in the Fteley and Stearns experiments on the 19-ft. weir, which Mr. Stearns once thought "quite satisfactory".* In fact, if one computes the velocities shown in Fig. 11 in the same manner in which Messrs. Fteley and Stearns presented the tabular results of their current-meter measurements in the channel of approach, one will find that the arithmetic mean of the individual departures from the average velocity in the section expressed as a percentage of the average velocity, is equivalent to 11% in the Fteley and Stearns channel of approach, whereas, the velocity distribution displayed in Fig. 11 gives a value of less than 5%, or more than twice as good a distribution as that once considered by Mr. Stearns as "quite satisfactory". Furthermore, the kinetic effect of the velocity of distribution shown in Fig. 11 is less than 3% smaller in effectiveness, in the production of "additional discharge due to velocity of approach", than that of a rigidly uniform distribution. A decrease of 3% in the velocity of approach correction is negligible, and represents the maximum effectiveness of screens or other devices which might have been added to the writer's flume in an effort to produce a still more uniform distribution of velocities. The effect of such screens would have been to produce a still greater discharge over the weir, instead of a smaller discharge, as Mr. Stearns seems to intimate.† For the reasons mentioned, there appears to be no foundation for Mr. Stearns' statement that, "judging from the high discharge over the author's weir with moderate heads, the conditions in his case were especially unfavorable". One is rather led to the opposite conclusion, namely, that conditions must have been especially favorable.

Fig. 25 involves velocity of approach rather than elimination of it, as Mr. Stearns would have one believe. The diagram may present a comparison of the different weir experiments with some degree of approximation, but its method of preparation is such that it does not present an infallibly true relation. In order to be absolutely correct, each individual experiment should have been treated with a velocity of approach correction differing from the others, in order to reduce the experiment to a channel of infinite depth. This correction is known only within certain limits and cannot be expressed by the mean value found by any experimenter, as these values $\left(\alpha \text{ in } h = \alpha \frac{v^2}{2g} \right)$ vary systematically for weirs of different heights and under different heads.

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 74.

† For a mathematical treatment of the effect of non-uniform distribution of velocities in the channel of approach, the reader is referred to *Water Supply Paper* No. 200, pp. 17 to 20.

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Messrs. Fteley and Stearns found that α varied from 1.33 to 1.87,* and adopted $\alpha = 1.5$ only as a mean value. The application of such a mean value of α to all the individual experiments, in an attempt to eliminate the effects of velocity approach, rather involves this factor, and presents the experiments in neither a proper relation to themselves nor to those of others. Likewise, Bazin found that α varied from 2.52 to 1.08, and, in presenting a method comparable with that adopted by Messrs. Fteley and Stearns, suggested the use of $\alpha = \frac{5}{3}$, stating, at the same time, that this method of correction for velocity of approach was not satisfactory. Bazin arrives at the following conclusion:

1st.—That the values of α decrease with decrease of height of weir, and that, for a given height, they first increase with the head, up to a maximum, and then again decrease.

2d.—That the values of K (in the Bazin formula), which vary less than those of α with the height of the weir, become more nearly equal as the head increases.

Furthermore, in his series of experiments on a weir 0.75 m. high, Bazin found a mean value for α equal to 2.21, which would seem to indicate that the application of $\alpha = 1\frac{2}{3}$ to the hydro-chemical experiments on a weir 1.135 m. high, in the preparation of Fig. 25, would give results with a deficient magnitude of correction, even in the average case. There is no assurance that $\alpha = 1\frac{2}{3}$ is the proper correction for any or all of the hydro-chemical experiments shown on Fig. 25. Furthermore, there is evidence that $\alpha = 1\frac{2}{3}$ does not show the individual Bazin experiments in their true location on this diagram. The same statement might also be made of the other experiments shown thereon. As an illustration of the amount to which the selection of the proper value of α affects the location of the experiments in the production of such a diagram as Fig. 25, the writer has prepared Fig. 27, showing the location of the curve representing the hydro-chemical experiments reduced to a channel of infinite depth with α equal to different selected values. Fig. 27 shows an exceptionally wide variation in range and, particularly, for the higher heads, a possible variation much in excess of the difference between the results of the various experimenters. It suffices also to show that Fig. 25 does not eliminate the effect of velocity approach, although it may present an excellent attempt to do so.

A much closer approximation to a correct presentation of a comparison of the different weir experiments with those presented in the writer's paper, has been prepared in Fig. 28. In this diagram, the actual coefficients in $Q = C\,L\,H^{\frac{3}{2}}$, for the hydro-chemical experiments, have been shown for the experimental conditions of velocity of approach

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), Table 1, p. 12.

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under which they were performed. The application of an unknown value of α is thus eliminated. The results of other experimenters have been reduced to like channel conditions, not by attempting to correct their individual experiments, but by reducing their derived formulas. Fig. 28 is merely a graphical presentation of the tabulation expressed in Table 4. It is similar to Fig. 25, except that it gives results for a channel 3.72 ft. deep. It presents a true comparison of the results of other experimenters with the writer's actual experiments. An inspec-

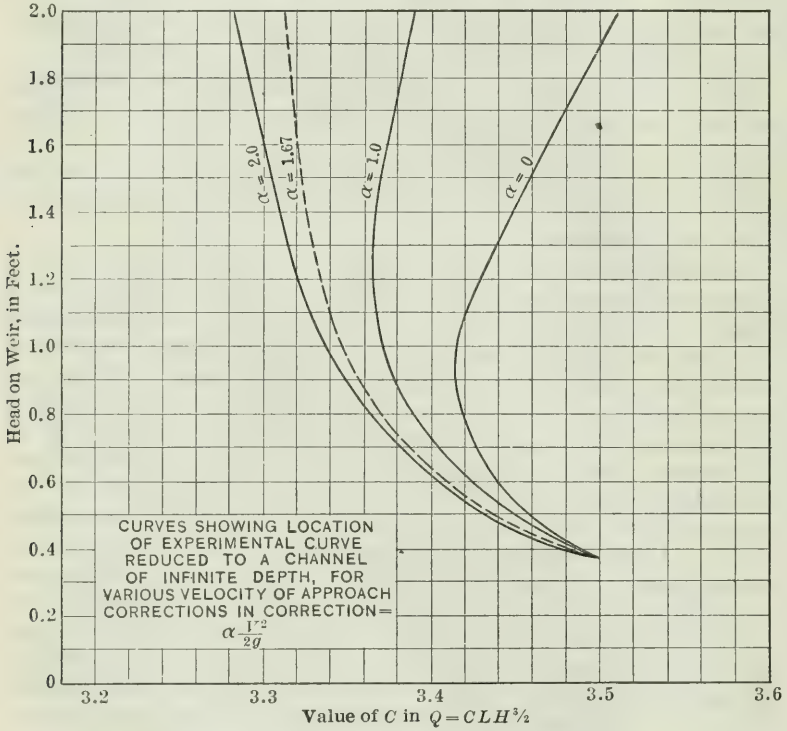

FIG. 27.

tion of Figs. 24 and 28 renders untenable Mr. Molitor's statement that in every case the Bazin experiments are smaller than those of the writer, and his conclusion that there is a constant or periodic error in the writer's experiments appears to be far from the truth. The hydro-chemical experiments do not give greater discharge than those of Bazin, except for the three experiments on the lowest heads, and even these fall within the limits of the Bazin experiments themselves. They verify the Bazin formula quite well for all heads, as well as the King formula and the Lyman diagrams for suppressed weirs, the latter two

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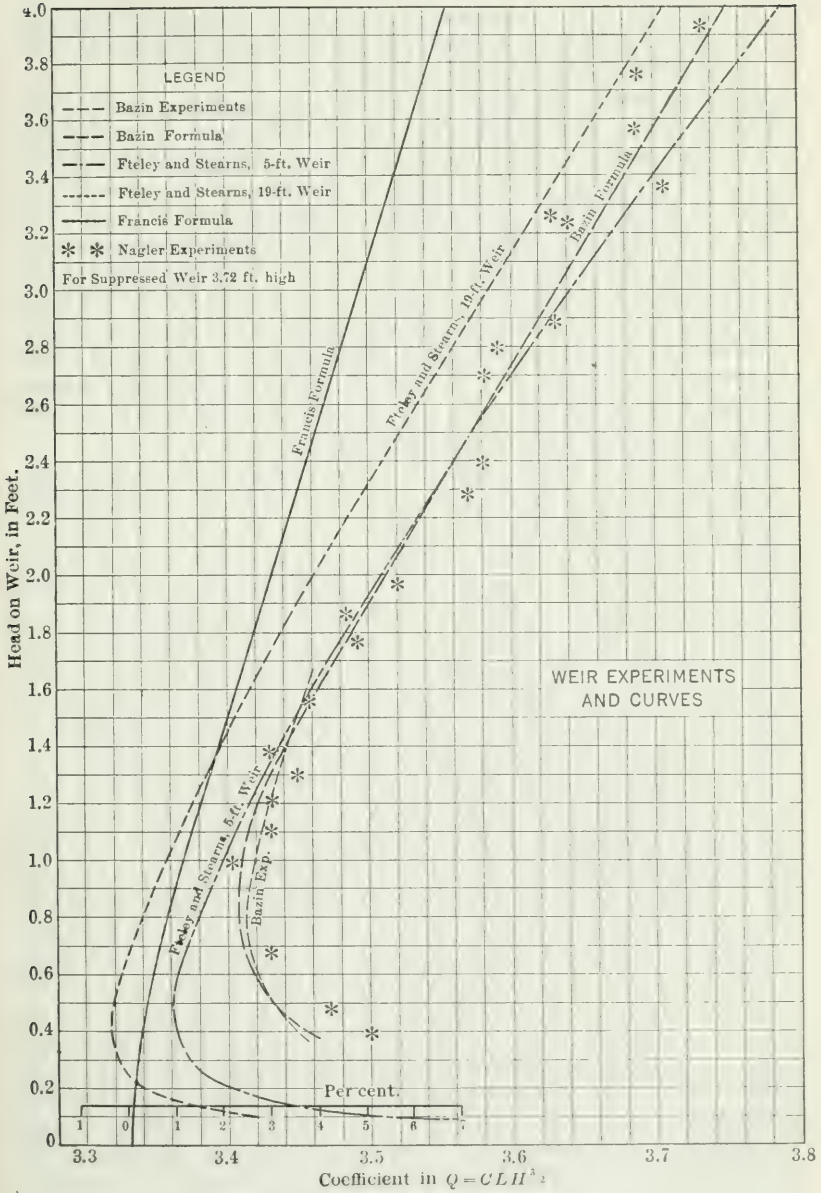

FIG. 28.

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having been recently derived after careful study of all standard weir experiments to date. Likewise, the writer is pleased to note the excellent agreement in the formula proposed by Gustav Ritter von Wex, presented in Mr. Molitor's discussion.

Some writers have mentioned the fact that the Bazin formula did not fit the Bazin experiments. This statement apparently ignores the fact that the Bazin formula was not based on the experiments on the standard weir alone, but on a total of 371 experiments on weirs of different dimensions, an aggregate number equal to more than all other so-called standard weir experiments combined. His formula does fit the entire series to a remarkable degree. If Messrs. Fteley and Stearns had attempted such a generalization of results, considerable difficulty would have been experienced in reconciling their two series of experiments on weirs of only two different dimensions, and their final result which they based on the work of Francis, as well as their own, departs more widely from its few basic experiments than the Bazin formula does from the many experiments from which it is derived.

In general, a few experiments confined to a limited range in conditions, no matter what precision may be claimed for them, do not form as good a basis for the evolution of a formula as a greater number distributed through varying conditions and over a wider range in head, the precision of which may be somewhat inferior than in the former case. If the few experiments of Messrs. Fteley and Stearns on the 19-ft. weir so well determined the weir formula, the writer is curious to know why they finally chose a general weir formula which departs widely from these experiments, fitting more closely those of Francis and their own experiments on the 5-ft. weir. The Rehbock formula furnishes another illustration of this same principle. The experiments were made on so small a scale (the weir being only 1.64 ft. long, from 0.41 to 1.64 ft. high, and with heads on the weir of from 0.03 ft. to 0.59 ft.), that it seems ridiculous to attempt to classify this work with large-scale experiments like those of Bazin, Francis, and Fteley and Stearns. The representation of the Rehbock experiments by the formula on Fig. 25 is quite misleading, since there are no experiments above a 0.59-ft. head on the weir and extended down to a 0.03-ft. head. A true representation of his work would be shown by a curve starting with 0.59 ft. on the weir, following that shown on Fig. 25, and extending beyond it in a downward direction across the entire chart, intersecting the right-hand boundary ($C = 3.6$), at about 0.055-ft. head. Mr. Stearns is also in error when he states that the Rehbock experiments above 1 ft. on the weir show very little change in coefficient with increasing heads, since Rehbock performed no experiments above 0.59-ft. head on the weir to indicate such a proposition at all; and the extension of the Rehbock formula to these higher limits is entirely unwarranted.

Mr. Nagler.

In contrast to the refined work of Bazin, in passing a given quantity of water over weirs of different lengths with side contractions suppressed, thus forming the basis whereby his formula gives the variation in C, which is greater than that of other experimenters, as shown by Mr. Stearns, the Tremont turbine tests involving the side contraction of weirs should have but little weight. In fact, if Mr. Stearns had used the correction for side contractions evolved by Francis for this very set of experiments, he would have been forced to quite a different conclusion. Mr. Francis* performed these experiments with the intention of determining the effect of side contractions, and developed the formula for these Tremont turbine tests, namely, $L = (l - 0.05\ nh)$, where L equals the corrected length; n, the number of contractions; l, the actual measured length of the weir; and h, the observed head. The change in the coefficient of the second term of the binomial from 0.1 to 0.05 quite materially affects the results in Table 7; and since Mr. Francis himself finally adopted 0.1 for the value of the coefficient merely because of its simplicity, the writer recomputed Table 7 on the more correct basis. The results show the "average difference" in the case of the Fteley and Stearns formula to be + 1.13%, in the case of the Rehbock formula, + 1.03%, and the Bazin experimental curve, — 0.43%. In other words, the Fteley and Stearns and Rehbock formulas do not increase as rapidly with decreasing heads on the weir by more than twice the amount which the Bazin experimental curve is found to exceed the proper rate of increase as indicated by the Tremont turbine experiments. A correct computation of these experiments, therefore, shows far better agreement with the Bazin formula than with the other two.

Mr. Molitor calls attention to the Rafter experiments, with heads on the weir as great as 4.67 ft., noting that they do not agree with the hydro-chemical experiments. Fig. 29 has been prepared to show a graphical comparison of the Rafter experiments with those of the writer. This diagram is similar to Fig. 28, being based on a weir 3.72 ft. high. The coefficients for the Rafter experiments which were on a weir 5.2 ft. high, have been reduced to equivalent coefficients for a weir 3.72 ft. high, by correcting the observed head by an amount equal to the difference in velocity of approach correction on weirs 5.2 ft. and 3.72 ft. high, respectively. A mean value of $\alpha = 2.2$ was used, being that found by Bazin in his experiments on velocity of approach for the weir most nearly equal to 3.72 ft. This is probably as close as the true correction could be approximated, and involves but little error, due to the fact that the discharge over the weir in the Rafter experiments was computed by the Bazin formula itself. The diagram reveals several facts: First, that the Rafter experiments are quite inconsistent within themselves, departing as much as 2% from the best curve which could be drawn through the series, even in instances

* "Lowell Hydraulic Experiments," p. 86.

Mr. Nagler.

at the lower heads; second, that there are only five experiments above 2 ft. in head, in contrast to eleven hydro-chemical experiments above this head; and these five high head experiments represent discharge coefficients from 1 to 5% less than the writer's experiments, coming into closer agreement for the higher heads; and third, that Experiments Nos. 6 and 12 in the Rafter series give higher coefficients than the hydro-chemical experiments, and, hence, Mr. Molitor's statement is not strictly accurate.

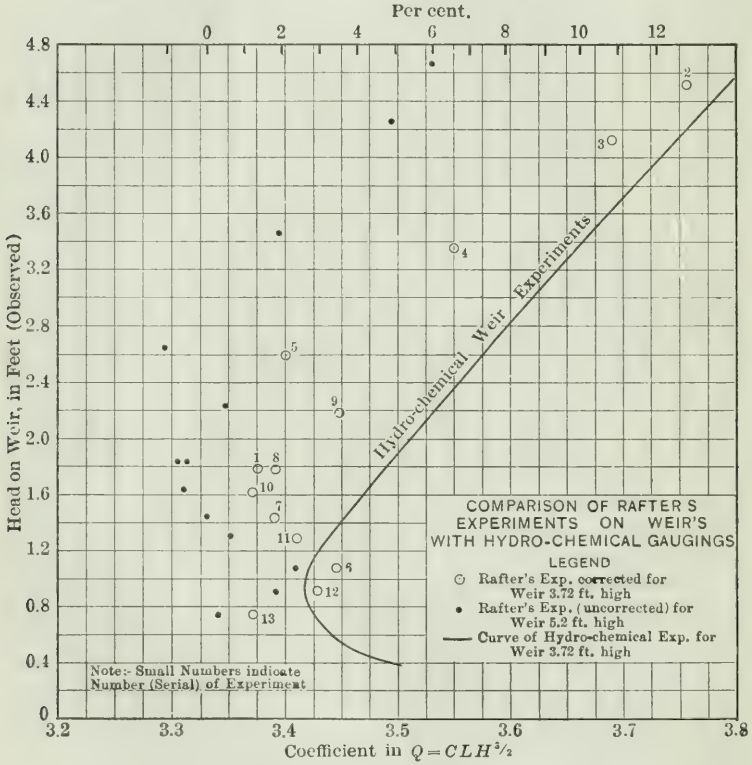

FIG. 29.

The Rafter experiments were in no sense volumetric experiments. A weir was used to gauge a weir of another length with different channel conditions; and it would hardly seem that they should be given much weight alongside of more accurate methods for determining the quantity of water passing over a weir. The discussion* of Mr. Rafter's work by Gardner S. Williams, M. Am. Soc. C. E., offers abundant proof as to the error in the Rafter experiments, error

* *Transactions*, Am. Soc. C. E., Vol. XXVI (1892), pp. 631 to 635.

sufficient, if allowed for, to make them come into close agreement with the hydro-chemical experiments. Mr. Nagler.

Mr. Molitor states that the hydro-chemical method of gauging has only high relative accuracy, and that its absolute accuracy must yet be proven. Can it be that the chemist has been mistaken and in processes of quantitative analysis has only been securing results of relative accuracy? The works of Collet, Stromeyer, and Mellet should afford some proof of the absolute accuracy of hydro-chemical gaugings. Mr. B. F. Groat has also presented some checks between hydro-chemical gaugings and volumetric and gravimetric gaugings, in which he found the two methods giving results agreeing within 0.2 and 0.3 per cent.* Mr. Molitor continues to mention sources of constant error which he fears may have existed in the writer's experiments. He mentions the effect of alkalis, acid, etc., in the pond water. The method of special dilutions in which laboratory solutions are made up in a series of trials, by mixing samples of the actual pond water and salt solution used in the test, absolutely eliminates all possibility of error from this source. Furthermore, if imperfect velocity distribution had affected the time that the salt reached the sampling station, its disastrous results would have become evident in a poor distribution of strength of samples. Table 2 shows not even moderate evidence of this, and also reveals that the whole mass of water as it passed the sampling section had practically the same concentration of salt. If differences had been observed in the samples, there might have been something more than speculation in Mr. Molitor's fears with regard to the location of sampling holes, peripheral flow, unequal distribution of velocities, and frictional losses in sampling pipes.

Water did not flow from the dosing station to the sampling point in imaginary stream lines, in which case the portions or elements retarded by friction would have arrived later; but there was interchange, many times, between the top and bottom and side elements, due to cross-currents and turbulence. This mixing was especially encouraged by the construction of baffles and a submerged weir in the flume between the two stations. The salt detection points in the water moving with low velocity, near the sides of the flume, proved that the salt solution traveled from the point of dosing to the sampling station, not with a velocity represented by the slow movement of the current in which the points lay, but equal to, and in some instances greater than, the mean velocity of the water in the flume. Finally, Mr. Molitor's last objection that there might be loss of salt by absorption in the flume walls, seems to be merely "straining at a gnat," when one considers that the flume walls are already soaked and filled with water to their capacity, and that, at best, the quantity of water absorbed in the walls represents only a small fraction of a percentage of the total volume of water discharged over the weir in any experiment.

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 1085, 1270.

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Paper No. 1428

DETERMINATION OF THE DUTY OF WATER BY ANALYTICAL EXPERIMENT*

By W. C. Hammatt, M. Am. Soc. C. E.

With Discussion by Messrs. J. C. Van Reigersberg Versluys, W. L. Rockwell, J. C. Stevens, S. T. Harding, and W. C. Hammatt.

Synopsis.

The purpose of this paper is to show the methods used in the determination of the quantity of water required for the growth of certain crops, where it was impracticable to measure it directly. This was done by analyses of the requirements and by the separate determination, by experiment, of the specific quantity required for the different parts which go to make up the whole use of water by the plant. The circumstances attending the investigation described herein were such as to open up a new field which is not covered to any great extent in available technical literature. The writer hopes that the description of his work in this case will bring forth discussion from others who may have treated this matter along similar or different lines, as this may broaden the scope of the literature on this very important subject.

The term "duty of water" has no hard and fast definition, and is considered by various investigators as representing different things. Some consider it to be the average annual quantity, in acre-feet, applied

* Presented at the meeting of March 20th, 1918.

by the farmers in a particular locality, or on a certain class of soil, to produce their crops. Others consider it as the minimum quantity which, if applied under certain conditions, will produce a crop. The writer considers it in these pages as the quantity which can be applied with the greatest efficiency, under given conditions, for the production of crops.

Many researches have been made in various States and agricultural localities by the Office of Experiment Stations or by State agricultural colleges, professing to obtain the duty of water for the particular localities and under the particular conditions obtaining in the area covered by the investigations. Among the most complete of these are those of Don H. Bark,* and the United States Department of Agriculture.† These investigations attempted to fix the economical duty by dividing each tract into three parcels, applying different quantities of water to each, and observing the results. This was a step ahead of any work done heretofore, but did not produce results sufficiently comprehensive, due to the great differences in the quantity of water applied and to the lack of a complete analysis of the reasons for the results obtained.

The experiments to be described later were for the purpose of determining the duty of water for the propagation of certain wild grasses in Southeastern Oregon. They were confined to two localities, the valley of the Chewaucan River, in Lake County, and the valley of the Silvies River, in Harney County. In both localities, crops of wild or native grasses are raised for forage purposes, the raising of beef cattle being the main industry. Climatic conditions in these two regions are such that crops of greater value cannot be raised economically. Both regions are subject to long and severe winters, and, even in the summers, killing frosts frequently occur, no month of the year being free therefrom. Although some garden truck is raised for local use, and there are occasional patches of alfalfa and grain, the raising of any of these on a commercial basis is a hazardous undertaking. The raising of native wild grasses, however, inured to the existing climatic conditions, is very profitable, such grasses having good food value, and the cattle raised thereon being famous for their quality.

* "Co-operative Investigations of Idaho."
† Reports for 1910, 1911, 1912, and 1913.

In the Chewaucan River Valley, Fig. 1, the river, on emerging from the mountains, flows on to a broad marsh, almost level transversely, and with a uniform fall longitudinally of about 1 ft. per mile. This marsh is divided into two parts by spurs of the mountain extending toward its center, leaving only a narrow channel between their extremities, through which the water passes from the upper to the lower end. This gives the marsh an hour-glass form, the area of the upper end being about twice as great as that of the lower, and

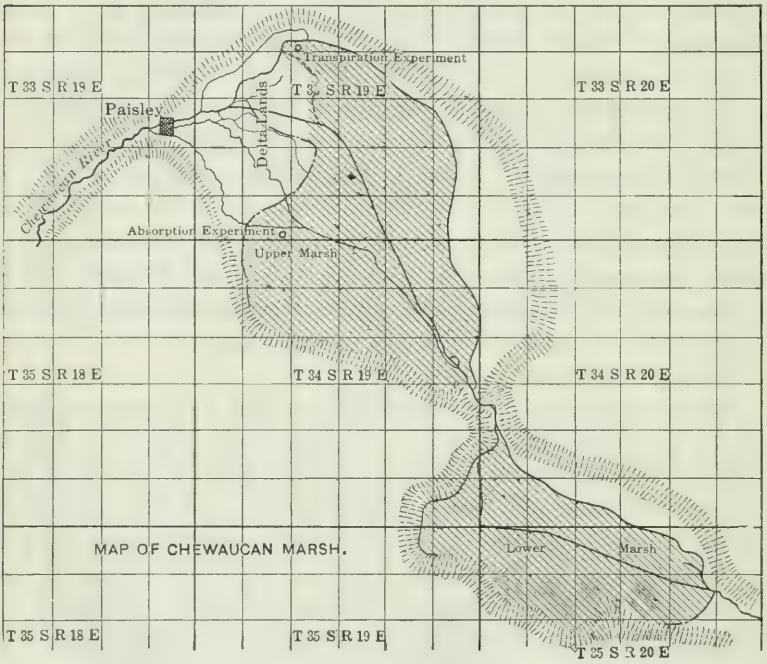
Transpiration Experiment
T 33 S R 19 E
T 33 S R 19 E
T 33 S R 20 E
Paisley
Delta Lands
Chewaucan River
Absorption Experiment
Upper Marsh
T 35 S R 18 E
T 34 S R 19 E
T 34 S R 20 E
MAP OF CHEWAUCAN MARSH.
Lower Marsh
T 35 S R 18 E
T 35 S R 19 E
T 35 S R 20 E

FIG. 1.

the two parts being designated the Upper and Lower Marshes, respectively. In former years, the spring freshets emerging from the mountains and discharging on the upper end of the upper marsh, spread indiscriminately over it, inundating it for long periods; the waters were gathered together again at the Narrows and at the lower end of this canyon, being again spread on the lower marsh, which they inundated before being again gathered together and discharged into Lake Abert, about 4 miles below. These inundations caused the growth of many plants of a somewhat aquatic nature, some of which were valuable for the

feed of animals and some of which were not. During the winter, this growth died down and was trampled by such stock as went on the marsh, so that a peat soil, from 1 ft. to several feet in thickness, has been built up. The nature of the inundations was such that full value could not be derived from the marsh for feeding purposes, on account of the likelihood of stock being trapped by freshets and lost by miring. The owner at that time, therefore, began to establish works for the purpose of controlling the water on the marsh and for facilitating drainage after the freshets ceased. These works have been amplified by successors in ownership, so that now there is quite a comprehensive system of works, designed to facilitate the even distribution of irrigation water and hold the freshet water off the marsh during such seasons as it would work injury rather than be beneficial for irrigation. The nature of the plants is such, due to the fact that they had their start under conditions of continuous flooding, that the same general method of applying water is still carried out. As will be described herein, however, the control of the water has caused an improvement in the nature of the plant growth, and will ultimately cause all the valueless plants to be replaced by those having a value for stock feed.

In the valley of the Silvies River (known as Harney Valley), Fig. 2, the river emerges from the mountains and splits up into various sloughs and branches, forming a delta over which the water has spread during times of freshet and annually causing the growth of certain wild grasses. Unlike the Chewaucan River, however, this is not a peat marsh, and the waters are not again brought together into one channel, but, after flowing over an area of considerable extent, are discharged into Malheur Lake. On account of the topography, the run-off is much more rapid, and the irrigation, consequently, is of shorter duration, so that the nature of the soil and of the crops is far different from those on the Chewaucan Marshes. To be sure, there are some pockets of small extent where conditions have been such as to cause the formation of peat layers and the growth of grasses similar to those on the Chewaucan Marshes, but, in the main, the growth is of a different nature.

Alkali.—In both the localities described, the valleys are surrounded by a volcanic formation locally known as the Rim Rock. The disintegration of the volcanic rocks releases alkaline salts, in greater or less

quantities, so that, though the water itself carries a very low proportion of such salts, the soils—of considerable porosity and capillarity—have accumulated them to a large extent by absorbing their quota of the flood water and later giving up the free water therefrom by soil evaporation. This characteristic of the soil limits and determines the nature of the plant growth which can thrive thereon, and also prevents any form of irrigation which does not provide for carrying away the alkaline salts which otherwise accumulate on the surface.

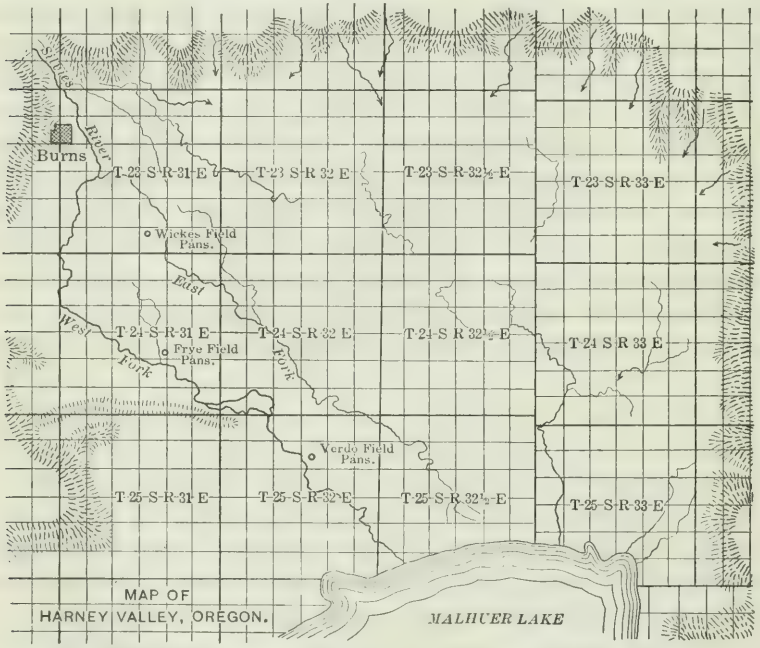
Burns
T 23 S R 31 E
T 23 S R 32 E
T 23 S R 32½ E
T 23 S R 33 E
Wickes Field Pans.
East Fork
West Fork
T 24 S R 31 E
T 24 S R 32 E
T 24 S R 32½ E
T 24 S R 33 E
Frye Field Pans.
Verdo Field Pans.
T 25 S R 31 E
T 25 S R 32 E
T 25 S R 32½ E
T 25 S R 33 E
MAP OF HARNEY VALLEY, OREGON.
MALHUER LAKE

FIG. 2.

Chewaucan Marsh is underlain beneath the peat with a chalky substance which absorbs water with great readiness, but resists its passage to a large extent. The nature of the subsoil is such that it is continually saturated, but permits of little or no loss by deep percolation. Even in continuous droughts, the ground does not dry to a great depth, as the subsoil gives up its water reluctantly.

The soil on the uplands at the head of the Chewaucan Marshes, which uplands are really a fan-shaped delta over which the waters of the river originally spread before flowing to the marsh, is of a different

nature. This soil is a fine, sandy loam, becoming more sandy with depth until the subsoil—of the same nature as that under the marsh—is encountered. In its general characteristics, the soil of the uplands above the Chewaucan Marsh is similar to that of Harney Valley.

Climate.—As already stated, the climate of Southeastern Oregon is very severe; winter lasts from October to March, the temperature often reaching many degrees below zero. The growing season is short, lasting only from April to September, and killing frosts are frequent in every month of the year. Two stations of the U. S. Weather Bureau are in the vicinity of Chewaucan Marsh, one at Paisley, 2½ miles above the marsh, and the other at Valley Falls, about 4 miles below it. The Weather Bureau station at Burns, Ore., at the head of the Harney Valley, provides data in regard to climatological conditions in that valley. For the purpose of indicating the nature of the climate in this vicinity, Tables 1 to 4 show the maximum, minimum, and mean temperatures for the stations at Paisley and Burns for a period of 11 years. Tables 5 and 6 show the maximum, minimum, and mean precipitation at the same stations for the same period of years.

TABLE 1.—Summary of Mean Monthly Temperatures Near Paisley. Compiled from U. S. Weather Bureau Records.

Year.	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1906..	33.2	33.1	35.8	47.0	51.1	55.6	71.4	66.5	58.6	48.5	38.1	36.8
1907..	27.0	42.4	37.4	48.4	52.9	55.8	65.4	60.2	55.5	52.0	....	....
1908..	32.0	36.0	39.8	48.2	48.2	57.6	71.3	66.3	59.8	47.0	41.8	28.2
1909..	35.6	37.4	40.5	46.8	51.0	60.4	64.4	66.4	50.3	50.7	41.2	27.0
1910..	28.0	33.2	46.2	53.2	57.9	59.2	69.6	65.4	56.5	50.8	38.8	34.0
1911..	30.8	27.3	42.0	44.6	48.7	61.4	77.5	....	53.8	47.0	37.8	30.4
1912..	37.0	40.2	38.6	....	....	59.6	....	....	....	....	....	31.6
1913..	28.4	31.2	37.8	45.2	....	57.3	63.6	65.8	57.6	49.0	44.8	27.9
1914..	....	....	....	44.2	54.4	58.5	67.0	67.3	54.8	47.6	....	21.0
1915..	30.4	34.4	42.6	48.0	48.6	57.1	63.8	70.8	56.0	51.5	36.4	....
1916..	24.0	39.0	42.2	45.7	46.6	56.0	62.5	65.6	58.3	46.6	34.8	27.1
Mean	30.6	35.4	40.3	47.1	51.0	58.0	67.1	66.0	56.1	49.1	39.2	29.3

Annual mean = 47.4.

Nature of Experiments.—As it was impracticable to segregate certain measured areas and determine their actual use of water, some other method had to be devised. After a careful study of conditions, the writer decided that the analytic method was probably the best, and would also lead to obtaining data which would be valuable in the determination of any changes of system which it might be desirable to make later for the purpose of improving irrigation conditions.

TABLE 2.—Comparison of Temperatures, from Records of U. S. Weather Bureau Stations, near Paisley and at Valley Falls, Oregon.

January.

	Paisley.			Valley Falls.		
Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	50	.....	.....	.....	.....	.....
1907	52	9	.....	.....	.....	.....
1908	49	7	.....	.....	.....	.....
1909	54	10	.....	.....	.....	.....
1910	56	— 9	.....	.....	.....	.....
1911	54	10	.....	.....	.....	.....
1912	57	5	.....	.....	.....	.....
1913	53	— 1	.....	.....	.....	.....
1914	.....	.....	.....	.....	.....	.....
1915	48	9	27	53	— 3	41
1916	47	6	20	55	— 2	32
	57	— 9	.....	55	— 3	.....

February.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	53	15	.....	.....	.....	.....
1907	60	23	.....	.....	.....	.....
1908	65	15	.....	.....	.....	.....
1909	53	12	.....	.....	.....	.....
1910	57	0	.....	.....	.....	.....
1911	45	10	.....	.....	.....	.....
1912	55	21	.....	.....	.....	.....
1913	62	5	.....	.....	.....	.....
1914	.....	.....	.....	57	11	.....
1915	50	19	24	56	15	38
1916	56	19	26	63	19	41
	65	0	.....	63	11	.....

March.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	60	— 3	.....	.....	.....	.....
1907	62	21	.....	.....	.....	.....
1908	67	19	.....	.....	.....	.....
1909	60	21	.....	.....	.....	.....
1910	67	23	.....	.....	.....	.....
1911	72	9	.....	.....	.....	.....
1912	60	21	.....	.....	.....	.....
1913	64	11	.....	.....	.....	.....
1914	.....	.....	.....	75	13	.....
1915	69	20	29	72	15	50
1916	71	12	38	78	15	49
	72	— 3	.....	78	13	.....

TABLE 2.—(*Continued*).

APRIL.

	PAISLEY.			VALLEY FALLS.		
Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	79	18	.....	.....	.....	.....
1907	71	23	.....	.....	.....	.....
1908	77	21	.....	.....	.....	.....
1909	70	24	.....	.....	.....	.....
1910	84	29	.....	.....	.....	.....
1911	70	18	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	72	22	.....	.....	.....	.....
1914	74	22	.....	75	22	.....
1915	78	25	40	82	20	50
1916	70	22	34	77	20	44
	84	18	.....	82	20	.....

MAY.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	81	29	.....	.....	.....	.....
1907	83	31	.....	.....	.....	.....
1908	74	26	.....	.....	.....	.....
1909	85	26	.....	.....	.....	.....
1910	92	31	.....	.....	.....	.....
1911	77	26	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	.....	.....	.....	.....	.....	.....
1914	86	27	.....	88	23	.....
1915	78	25	37	81	14	44
1916	79	20	36	84	15	52
	92	21	.....	88	14	.....

JUNE.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum	Range.
1906	80	34	.....	.....	.....	.....
1907	83	28	.....	.....	.....	.....
1908	86	32	.....	.....	.....	.....
1909	89	37	.....	.....	.....	.....
1910	91	32	.....	.....	.....	.....
1911	89	33	.....	.....	.....	.....
1912	87	34	.....	.....	.....	.....
1913	84	35	.....	.....	.....	.....
1914	95	29	.....	93	25	.....
1915	87	31	35	90	28	52
1916	89	30	39	93	25	51
	95	28	.....	93	25	.....

TABLE 2.—(*Continued*).

JULY.

	Paisley.			Valley Falls.		
Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	92	50	.....	.....	.....	.....
1907	89	40	.....	.....	.....	.....
1908	95	48	.....	.....	.....	.....
1909	89	35	.....	.....	.....	.....
1910	95	42	.....	.....	.....	.....
1911	95	39	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	88	34	.....	.....	.....	.....
1914	93	40	.....	96	33	.....
1915	93	38	49	98	33	52
1916	91	36	40	94	29	55
	95	34	.....	98	29	.....

AUGUST.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	80	40	.....	.....	.....	.....
1907	87	33	.....	.....	.....	.....
1908	94	33	.....	.....	.....	.....
1909	93	41	.....	.....	.....	.....
1910	90	36	.....	.....	.....	.....
1911	.....	.....	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	91	35	.....	.....	.....	.....
1914	96	38	.....	99	32	.....
1915	97	49	39	102	40	57
1916	91	31	41	95	26	56
	97	31	.....	102	26	.....

SEPTEMBER.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	86	30	.....	.....	.....	.....
1907	86	33	.....	.....	.....	.....
1908	88	24	.....	.....	.....	.....
1909	84	26	.....	.....	.....	.....
1910	82	31	.....	.....	.....	.....
1911	81	35	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	88	27	.....	.....	.....	.....
1914	80	29	.....	88	22	.....
1915	82	32	34	90	20	52
1916	83	33	36	90	24	57
	88	24	.....	90	20	.....

TABLE 2.—(*Continued*).

OCTOBER.

	PAISLEY.			VALLEY FALLS.		
Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	80	23	.....	.....	.....	.....
1907	69	27	.....	.....	.....	.....
1908	76	19	.....	.....	.....	.....
1909	74	31	.....	.....	.....	.....
1910	78	30	.....	.....	.....	.....
1911	74	26	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	94	19	.....	.....	.....	.....
1914	72	30	.....	85	19	.....
1915	80	27	37	85	15	60
1916	74	24	32	75	10	53
	94	19	.....	85	10	.....

NOVEMBER.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	67	10	.....	.....	.....	.....
1907	.....	.....	.....	.....	.....	.....
1908	67	19	.....	.....	.....	.....
1909	65	11	.....	.....	.....	.....
1910	66	7	.....	.....	.....	.....
1911	58	14	.....	.....	.....	.....
1912	.....	.....	.....	.....	.....	.....
1913	67	19	.....	.....	.....	.....
1914	60	.....	.....	69	5	.....
1915	57	14	26	71	12	48
1916	61	5	28	72	— 6	60
	67	5	.....	72	— 6	.....

DECEMBER.

Year.	Maximum.	Minimum.	Range.	Maximum.	Minimum.	Range.
1906	58	6	.....	.....	.....	.....
1907	.....	.....	.....	.....	.....	.....
1908	50	— 4	.....	.....	.....	.....
1909	50	2	.....	.....	.....	.....
1910	.....	0	.....	.....	.....	.....
1911	50	10	.....	.....	.....	.....
1912	54	9	.....	.....	.....	.....
1913	50	10	.....	.....	.....	.....
1914	47	0	.....	48	12	.....
1915	.....	0	.....	58	— 9	40
1916	48	3	34	55	— 2	35
	58	— 4	.....	58	—12	.....

TABLE 3.—Summary of Mean Monthly Temperatures near Burns, Oregon.

Compiled from U. S. Weather Bureau Records.

Year.	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1906..	23.4	23.6	29.2	47.0	49.1	58.6	72.8	67.7	56.6	57.6	29.4	31.2
1907..	23.2	39.6	38.2	50.5	54.6	55.5	67.4	60.8	54.8	54.4	37.0	29.6
1908..	26.5	32.8	40.1	49.4	44.5	68.2	69.6	64.3	57.6	45.3	38.3	24.4
1909..	28.9	31.8	38.7	41.9	49.0	59.2	66.1	66.2	57.6	44.8	37.2	16.4
1910..	17.5	25.4	42.6	49.2	55.2	56.9	68.9	63.0	56.9	49.4	36.8	26.6
1911..	25.3	18.0	38.3	41.8	45.9	60.4	67.6	63.1	51.4	43.2	35.8	23.8
1912..	25.2	32.4	41.7	42.8	51.6	58.0	....	64.1	58.0	43.9	36.3	25.2
1913..	20.4	25.8	35.2	44.0	54.4	60.4	68.6	68.6	57.8	46.7	39.8	26.5
1914..	....	30.0	45.9	48.2	53.7	53.1	68.8	....	....	....	31.6	12.9
1915..	19.6	32.6	42.5	54.6	50.4	58.1	64.9	73.4	61.5	....	33.6	....
1916..	....	....	43.0	47.2	....	....	....	....	....	....	....	....
Mean	23.3	29.2	39.6	47.0	50.8	58.8	68.3	65.7	56.9	47.4	35.6	24.1

Annual mean = 45.6

The analytic method consisted in dividing the water into its various uses and making a separate determination, as far as possible, of each use. In this manner, any of the determinations which were based on estimates, or for which the data were inadequate or unreliable, would affect only the particular item concerned, and not the whole duty of water. Another advantage of this method of determination is that, by a study of the different factors making up the analysis, unnecessary or reparable losses may be discovered and the system improved so as to eliminate them.

In a strict sense, the only water actually used by a plant in its growth is that taken in through the roots and either remaining as a component part of it or being given off through its leaves by the process of transpiration. However, as it is impossible to deliver water to the roots directly, and as, even if it were possible to do so, it would be necessary to pass the water through the soil in order that it might take into solution the salts necessary for plant growth, there are various other uses which can be considered as beneficial to plant growth. Consequently, it may be considered that the various components into which the beneficial use of water is divided are as follows:

(1) Soil Moisture,
(2) Soil Evaporation,
(3) Plant Use,
(4) Soil Losses.

TABLE 4.—Range of Temperature at U. S. Weather Bureau Station at Burns, Oregon.

Year.	January.			February.			March.			April.		
	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.
1906	45	— 4	....	48	— 7	....	54	—12	....	70	24	....
1907	47	— 8	....	59	20	....	66	16	....	78	24	....
1908	46	— 8	....	65	9	....	66	12	....	80	18	....
1909	49	— 7	....	56	9	....	66	18	....	68	12	....
1910	46	-24	....	52	— 9	....	68	21	....	85	19	....
1911	50	— 9	....	39	7	....	79	7	....	70	12	....
1912	47	—21	....	49	6	....	71	13	....	72	20	....
1913	48	—13	....	49	0	....	64	8	....	79	13	....
1914	..	3	....	54	— 1	....	74	14	....	74	23	....
1915	44	—20	38	47	9	30	72	14	45	85	24	60
1916	..	..	....	..	....	....	70	..	....	76	..	....
	50	—24	....	65	— 9	....	79	—12	....	85	12	....

Year.	May.			June.			July.			August.		
	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.
1906	81	25	....	90	31	....	97	42	....	96	37	....
1907	83	30	....	85	30	....	97	40	....	95	32	....
1908	76	25	....	88	30	....	100	38	....	96	27	....
1909	84	22	....	89	30	....	95	31	....	100	31	....
1910	97	28	....	94	25	....	100	37	....	95	33	....
1911	78	17	....	97	31	....	97	29	....	93	34	....
1912	80	25	....	89	29	....	..	36	....	100	25	....
1913	89	26	....	90	31	....	95	34	....	100	34	....
1914	86	26	....	86	26	....	96	34	....	94	40	....
1915	77	27	....	89	34	49	99	34	....	99	48	....
1916	..	..	....	..	..	....	..	..	....	..	..	....
	97	17	....	97	25	....	100	29	....	100	25	....

Year.	September.			October.			November.			December.		
	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.	Max.	Min.	Range.
1906	87	26	....	84	16	....	58	— 2	....	50	5	....
1907	92	26	....	81	27	....	64	15	....	59	4	....
1908	92	17	....	79	19	....	66	9	....	41	— 2	....
1909	89	24	....	71	19	....	59	5	....	44	—11	....
1910	87	28	....	84	15	....	64	— 2	....	50	0	....
1911	90	19	....	66	17	....	61	6	....	54	— 6	....
1912	90	26	....	82	12	....	60	14	....	48	— 2	....
1913	89	26	....	72	16	....	70	10	....	52	— 1	....
1914	..	..	....	..	..	....	56	5	....	44	—13	....
1915	91	33	....	..	..	....	58	— 4	40	..	....	....
1916	..	..	....	..	..	....	..	....	....	..	....	....
	92	17	....	84	12	....	70	— 4	....	59	—13	....

TABLE 5.—Summary of Mean Monthly Precipitation near Paisley.

Compiled from U. S. Weather Bureau Records.

Year.	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1906....	....	....	....	0.44	1.11	0.55	1.03	0.94	0.15	0.89	0.75	0.89
1907....	1.03	1.67	2.65	0.09	1.62	2.80	0.30	0.50	1.35	0.88	....	....
1908....	0.34	0.58	0.11	0.21	0.95	0.56	1.07	0.44	0.40	1.42	0.41	0.30
1909....	2.60	0.55	1.02	T.	0.91	0.81	0.23	0.06	0.81	0.98	2.06	0.77
1910....	0.30	0.49	0.67	0.21	0.12	0.49	0.30	0.00	0.76	0.29	2.83	0.18
1911....	2.95	1.33	0.53	1.24	0.10	1.83	T.	....	1.31	0.45	0.14	T.
1912....	2.44	1.83	0.57	2.30	2.78	2.36	0.82	0.49	0.75	0.82	0.94	2.25
1913....	0.89	0.18	0.07	0.34	....	1.95	2.34	0.79	0.42	0.11	1.38	0.80
1914....	....	....	....	1.50	1.36	2.25	0.69	T.	1.59	2.51	0.51	0.39
1915....	0.60	2.89	1.51	1.02	1.72	0.23	1.56	0.06	0.08	0.57	2.54	2.46
1916....	2.46	1.35	2.21	1.35	2.83	1.53	0.42	0.40	0.61	0.08	1.67	1.54
Mean..	1.51	1.21	1.04	0.79	1.35	1.40	0.80	0.37	0.76	0.82	1.32	0.96

Mean annual precipitation = 12.32.

TABLE 6.—Summary of Mean Monthly Precipitation near Burns.

Compiled from U. S. Weather Bureau Records.

Year.	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1906....	4.24	1.38	3.94	0.26	2.43	1.41	0.36	0.43	0.59	0.23	0.96	1.88
1907....	2.08	2.36	2.33	1.06	0.51	1.59	0.26	0.10	1.01	0.17	0.51	3.49
1908....	0.90	0.23	0.34	0.71	1.05	0.73	0.42	0.31	0.81	1.35	0.48	0.59
1909....	3.86	1.54	0.64	0.10	0.51	0.23	0.07	0.00	0.90	1.01	2.95	2.29
1910....	1.29	1.97	1.74	0.32	0.34	0.04	0.03	0.00	0.83	1.16	3.33	1.27
1911....	1.59	0.50	0.18	0.36	0.75	0.56	0.13	0.00	0.55	0.13	0.72	0.71
1912....	1.94	1.35	0.55	1.97	1.69	1.17	0.91	0.44	0.61	0.16	1.66	1.19
1913....	1.55	0.15	0.86	0.99	0.25	2.93	1.99	0.19	0.15	1.63	1.08	2.05
1914....	2.52	0.80	0.15	1.49	0.94	1.41	0.00	0.00	....	....	0.03	0.42
1915....	1.06	1.60	0.90	1.18	0.94	0.40	0.90	0.06	0.04	....	1.26	....
1916....	....	....	0.38	0.44	....	....	....	....	.. .	....	....	....
Mean..	2.10	1.18	1.09	0.80	0.94	1.04	0.50	0.15	0.61	0.73	1.29	1.54

Mean annual precipitation = 12.01.

(*1*).—*Soil Moisture.*—The necessary soil moisture differs in different cases. Some plants require a soil having little more than hygroscopic moisture; from this minimum the requirements of various plants increase through all gradations, some even requiring a saturated soil with free surface water. Among the latter, the most widely known is rice.

(*2*).—*Soil Evaporation.*—As water cannot be brought through the soil to the roots of plants without a certain proportion being drawn to the surface by capillarity and thence evaporated, soil evaporation may be considered as a beneficial use. This evaporation will vary

according to the method of irrigation and the nature of the plant—deep-rooted plants irrigated by sub-surface application requiring very little soil evaporation, and those which must have a saturated soil and free surface water requiring a very high evaporation.

(*3*).—*Plant Use.*—Plant use consists of the water going into the plant structure and that given off by transpiration through the leaves. The first is negligible when compared with the other components. Transpiration, however, is one of the most important uses of water.

(*4*).—*Soil Losses.*—Certain losses in the application of water are necessary, for various reasons: Few subsoils permit of the application of water by any of the irrigation methods in general use without deep percolation or subsoil losses. The nearer to saturation the soil is required to be, the greater will be the underground losses. Where free water is required on the surface, a certain quantity of this must be drained off, in order that continuous evaporation will not cause the concentration of the injurious salts carried. In some cases, a movement of water is also required in order to avoid the loss of air from the water and the consequent stagnation which is injurious to such plants. This movement necessarily entails an additional loss, that of run-off.

To the foregoing may also be added delivery losses in bringing the water to the land. In this paper, however, these losses are not considered, as the duty of water is here treated as the net duty on the land, and not the gross duty from the point of supply.

The growth on the Chewaucan Marshes consists of certain grasses of local importance which are little known in other parts of the country. The most valuable of these are the "Awned Sedge" (*Carex Aristeta*) and "Sprangle Top" (*Scholochloa Festucacea*). There is very little literature on these two grasses, and their habitat seems to be extremely limited. Both occur only in severe climates, no examples being known south of the forty-second parallel of latitude, and they are found on the North American Continent only in Canada and some of the most northerly parts of the United States. They grow in profusion in a few places in Oregon, and all that have been examined seem to require a peat soil and conditions of inundation during the growing season. Apparently, a distinct movement of the water is also necessary, as any check in the flow seems to cause a replacement of these grasses by tules, cat-tails, and other waste growths which thrive

in still or stagnant water. Where conditions are right, in regard to the period of inundation and the flow of water, and where the soil is a true peat, the plants attain a height of from 4 to 8 ft., cutting from 2½ to 5 tons of cured hay to the acre. Where water is plentiful, and the inundation is continuous during the growing season, neither of these grasses flowers, but the growth goes entirely to stalk and leaf. In this condition, the appearance of both kinds is very similar, so that only a person familiar with the characteristics of the two plants is able to distinguish one from the other. They are locally known as "Sugar Grass", the term being applied indiscriminately to the two varieties. The two occur sometimes in mixed patches, and sometimes in patches containing only one of the varieties.

Where the water supply fails before the culmination of the growth of these plants, and where such plants reach maturity under conditions of a moist soil, instead of a saturated soil with surface water running over it, they sometimes go to seed, particularly the Sprangle Top. This condition obtains on various high knolls which do not receive as much water as the remainder of the marsh, and the appearance of the plant in flower is so different from that before flowering that it has been popularly supposed to be of a different variety, and is locally known as "Rice Grass".

There have been no exact experiments to determine the food value of the plant, either by feeding to stock or by analysis of the hay. Among the ranchers conversant with this subject, however, it is generally conceded that, pound for pound, Sugar Grass hay is equal to Alfalfa for the fattening of beef cattle. There is a difference of opinion as to its value for feeding work stock, some holding that work horses hold up slightly better on Timothy hay, and some claiming that Sugar Grass hay is equal or superior to Timothy hay for work stock as well as beef cattle. The fact remains, however, that the company operating the marshes feeds more than 10 000 head of cattle for 5 winter months on 20 lb. of Sugar Grass hay per day per animal, bringing them out in the spring in much better condition than they were when they went in in the fall, and also that, during the harvest season, both the cow-ponies and the draft animals on the harvesting machinery are worked under extremely severe conditions on a diet consisting only of Sugar Grass hay, without a pound of grain, which shows that the food value is excellent.

FIG. 3.—NORMAL GROWTH OF SUGAR GRASS.

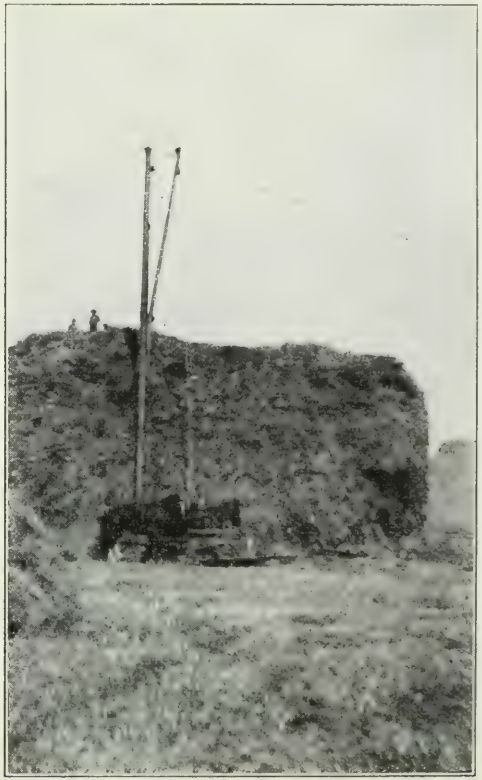

FIG. 4.—STACKING SUGAR GRASS HAY.

FIG. 5.—FIELD OF SUGAR GRASS DURING CUTTING.

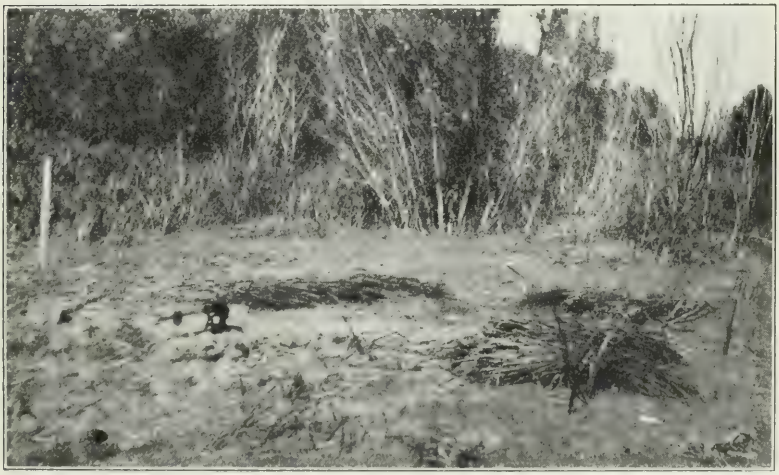

FIG. 6.—TRANSPIRATION EXPERIMENT BY CUTTING.

The vegetation in Harney Valley consists of a miscellaneous growth of wild or partly wild grasses. A large part of the land was seeded in the past to Timothy and Red-top, and traces of these grasses remain throughout much of the valley. Mixed with them, however, on the sedimentary soils, are found "Nevada Blue Grass", "Soft Leaf

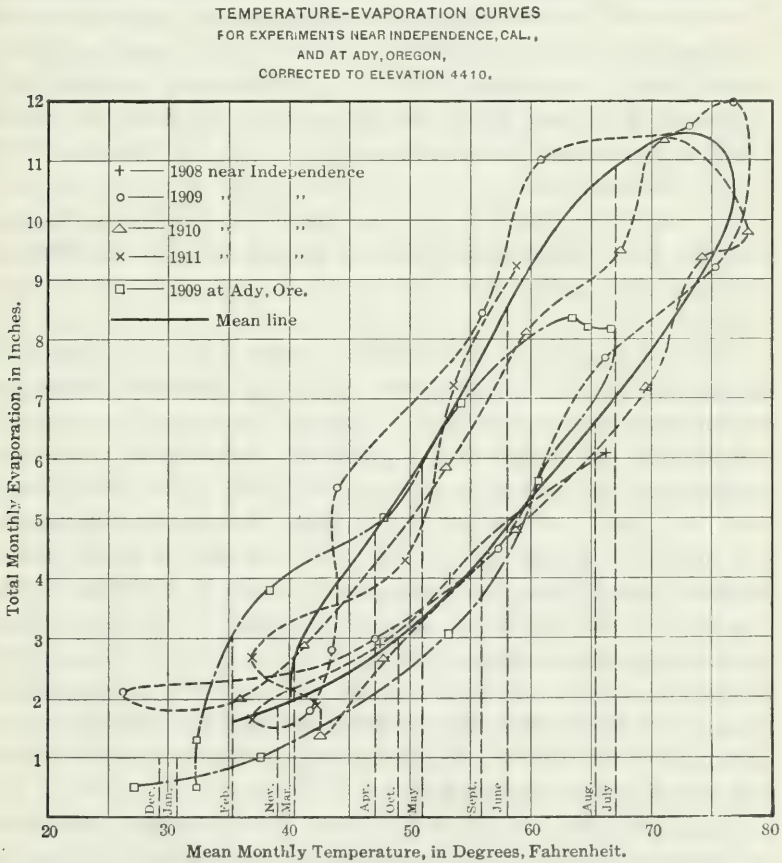
TEMPERATURE-EVAPORATION CURVES
FOR EXPERIMENTS NEAR INDEPENDENCE, CAL.,
AND AT ADY, OREGON,
CORRECTED TO ELEVATION 4410.
+ —— 1908 near Independence
o —— 1909 " "
△ —— 1910 " "
× —— 1911 " "
□ —— 1909 at Ady, Ore.
—— Mean line
Total Monthly Evaporation, in Inches.
Mean Monthly Temperature, in Degrees, Fahrenheit.
Dec. Jan. Feb. Nov. Mar. Apr. Oct. May Sept. June Aug. July

FIG. 7.

Sedge" and other less valuable wild grasses. Where peat soil occurs, heavy growths of "Sugar Grass" are found.

Evaporation.—Very few evaporation measurements have been taken in Oregon, and the conditions attending these investigations have been such as to make it impossible to carry on continuous experiments through all parts of the year. Continuous evaporation experi-

ments, however, have been made in various places in California, and those in Owens Valley by C. H. Lee, M. Am. Soc. C. E., during the period from 1908 to 1911, under the auspices of the Los Angeles Aqueduct Commission and the U. S. Geological Survey,* were taken as most nearly conforming to conditions as they occur in Eastern Oregon. The elevation of Owens Valley above sea level at the point where the Lee experiments were undertaken is 3 775 ft.; the elevation of Chewaucan Marsh is approximately 4 410 ft. Temperature conditions for Chewaucan Marsh are shown by Tables 1 and 2; those for Owens Valley were obtained in a similar manner from the records of the U. S. Weather Bureau.

The method of deducing the evaporation for the various months from free water surface near Paisley is shown by Fig. 7. In order to understand this matter thoroughly, a few preliminary points should be discussed.

The measurements of evaporation in Owens Valley were corrected for altitude in the inverse proportion which the barometric heights in the two places bore to each other. Although it has never been proved, academically, that evaporation at different elevations is in inverse proportion to the barometric pressure, the writer's observations have always led him to believe this to be a fact. Relative to this matter, it is interesting to note that, in the paper entitled "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico",† by Edwin Duryea, M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E.,‡ about a year after the experiments described in this paper, the authors tabulate the results of all evaporation measurements throughout a broad area in the Southwestern United States and Northern Mexico in order to show the effect of elevation, and this tabulation shows indubitably that it is as above stated.

For a barometric height of 30 in. at sea level, that at Elevation 3 775 would be 26 in., and that at Elevation 4 410 would be 25.35 in., from which it is seen that the evaporation at Elevation 4 410, all other conditions being equal, would be 16% greater than that at Elevation 3 775.

Professor Frank H. Bigelow and others have done considerable investigatory work to determine the relation between the evaporation

* Published in *Water Supply Paper No. 294.*

† *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), p. 1829.

‡ Now M. Am. Soc. C. E.

from pans of various diameters and from lake surfaces. In the writer's experiments no consideration has been given to the possible lesser evaporation from a lake surface than from the pans used by Mr. Lee, due to the fact that conditions of cover were such as to make the evaporation more nearly conform to that found in pans than to that of a broad, uninterrupted lake surface.

In comparing the evaporation in Owens Valley with that in the places covered by this investigation, allowance must be made for differences in humidity. Although no data are obtainable as to the relative humidity in the two locations, observation on the ground and a study of the topographical conditions enables one to make an approximate allowance. The Chewaucan Marsh is enclosed by high bluffs which protect it, more or less, from the action of the wind, and cause the humidity during times of flood to be higher than in broad valleys like those of the Owens and Silvies Rivers, the latter two being somewhat similar in configuration, and humidity conditions being more nearly the same. Moreover, the growing season in the Owens River Valley is longer than in Oregon, but the proportion of flooded land is very much smaller, which would tend to make the humidity, during the early months of the year and also during the summer, higher in the Owens Valley than in the Oregon valleys.

On Fig. 7 is platted a curve in which the abscissas represent the mean monthly temperatures, in degrees, Fahrenheit, and the ordinates the total monthly evaporation, in inches, for each month from September, 1908, to May, 1911, corrected for difference in elevation between 3 775 and 4 410 ft. From this, a mean curve is constructed, shown by the full line. In Table 9 are given, first, the results of applying the mean monthly temperatures near Paisley to this curve and getting the corresponding evaporation by intersection with the mean curve; second, the assumed mean monthly evaporation near Paisley after making corrections for the difference in local conditions; and third, the monthly evaporations obtained by intersecting the curve deduced from evaporation measurements at Ady, Ore., by the mean monthly temperatures for the station near Paisley. The latter evaporation measurements were taken under the direction of the U. S. Weather Bureau, in 1909, at Ady, Ore., near Klamath Lake, at an elevation of 4 090 ft., and the curve platted on Fig. 7 is deduced from these measurements, correcting for the difference of elevation by increasing the

TABLE 7.—Monthly Evaporation Record near Independence, California.

Elevation, 3 735. 3-Ft. Square Pan.

Month.	1908		1909		1910		1911	
	Evaporation.	Mean temperature.	Evaporation.	Mean temperature.	Evaporation.	Mean temperature.	Evaporation.	Mean temperature.
January	.....	.....	1.60	41.7	1.75	35.6	1.65	42.4
February	.....	.....	2.40	43.6*	2.50	41.4	2.35	36.7
March	.....	.....	4.70	44.2	5.15	53.2	3.70	49.8
April	.....	.....	7.30	56.0	7.05	59.8	6.25	53.6
May	.....	..	9.60	60.8	8.29	67.5	8.01	58.7
June	.....	.....	10.10	73.0	9.90	71.0*	....	.....
July	.....	.....	10.40	76.6	8.50	77.9	....	.....
August	.....	.....	8.00	75.3	8.20	74.2*	....	.....
September	5.30	66.2	6.60	66.2	6.30	69.7	....	.....
October	3.50	53.4	3.90	57.3	4.20	58.6	....	.....
November	2.50	47.2	2.60	47.0	2.36	48.0	....	.....
December	1.50	36.6	1.85	26.0	1.24	42.4*	....	.....
Totals	.....	.....	69.05	55.6	65.44	58.3	....	.....

Evaporation from Water Supply Paper No. 294.

Temperature from Weather Bureau records at Independence.

* Record at Lone Pine.

TABLE 8.—Monthly Evaporation Record at Ady, Oregon.

Elevation, 4 090. 4-Ft. Cylindrical Pan.

1909.

Month.	Evaporation.	Mean temperature.
January	0.50*	32.2†
February	1.25*	32.5†
March	3.57*	38.4†
April	4 64	47.7
May	6.48	54.2
June	7.78	63.6
July	7.65	64.6
August	7.64	66.6
September	5.26	60.5
October	2.87	53.2
November	1.00*	37.5†
December	0.50*	27.0†
Totals	49.14	48 2

* From Moritz's "Working Data for Irrigation Engineers."

† From Weather Bureau records at Klamath Falls.

Other data from Weather Bureau records at Ady, amplified by interpolation for missing days.

TABLE 9.—Evaporation from Water Surface near Paisley, Oregon.

Elevation, 4 410.

Month.	Mean monthly temperature.	Monthly evaporation, from chart.	Monthly evaporation, assumed.	Monthly evaporation, from Ady line.
January	30.6	No intersection.	1	No intersection.
February	35.4	" "	1	2.9
March	40.3	2.9	2	4.1
April	47.1	4.7	3	4.9
May	51.0	5.9	4	5.7
June	58.0	8.4	7	7.6
July	67.1	10.8	8	8.2
August	66.0	6.6	7	7.1
September	56.1	4.3	5	3.7
October	49.1	3.0	3	2.4
November	39.2	1.9	1	1.2
December	29.3	No intersection.	1	0.6
Totals	47.4	....	43	....

evaporation measurements 7½ per cent. These measurements were not available at the time the original computations were made, but, on making use of them, at present, it is interesting to note how closely they conform to the evaporation assumed, and how well the variations from the assumed ones are accounted for by the equalizing influence of Klamath Lake.

Having obtained the evaporation from the water surface by use of the diagram, Fig. 7, it was necessary to reduce this to the condition of evaporation from a thin sheet of water passing over the soil and to conditions of protection by plant growth.

The first correction was made by adding 33% to the evaporations obtained, as described for the water surface, as investigators have found that the evaporation from a thin sheet of water is greater than that from an ordinary water surface, varying between 25 and 37 per cent. As for the plant protection, conditions are such, due to the close growth of the plants, that the humid air is held impounded by their leaves without being swept away by the wind and replaced by dry air, thus making the evaporation lower than it would be for an unprotected water surface. In these experiments, therefore, it was assumed that the evaporation from a field containing fully developed plants would be only one-third of that from the same field without vegetation. The ratio of evaporation, therefore, was assumed to drop uniformly from its full quantity at the beginning of the growing sea-

son to one-third of that quantity at the end of that season, and the evaporation above determined was corrected in this manner. Therefore, the evaporation is corrected to conditions of plant cover, as follows:

March 15th to 31st	1.33 in.
April	3.11 "
May	2.96 "
June	3.11 "
Total	10.51 in.

In the Harney Valley analysis, time was available for more complete experimental work than was done on the Chewaucan Marshes. Pan experiments were made in order to determine evaporation in connection with transpiration, and, as the records of soil evaporation were thus directly determinable, no analysis was made for this valley by comparison with distant data.

The pans used for evaporation tests were 11.3 in. in diameter, and 4 ft. deep, having a surface area of 100 sq. in., and a soil depth of 3 ft. inside the pan. The pans were sunk in the soil to a depth of 3 ft., and the soil was replaced inside them in the same order as it was taken out of the pit, all vegetation, however, being removed from the surface in order that the record of the pan would give the soil evaporation without being affected by transpiration. The diameter of the pan appears to be a trifle small, but it was constructed in that way in order that its surface would receive, as nearly as possible, the same protection as the surrounding soil. A gauging well of small diameter was put in the pan in order that the depth to ground-water could be determined at intervals. Observations were made every few days, and enough water was added to bring the ground-water elevation to the same height as that in the natural soil outside the pan, the water being measured into the pan to replace evaporation losses.

At the end of the test, at harvest time, enough water was added to bring the ground-water level to the same height as that at the beginning of the test, the total water added thus equalling the evaporation during the period of the tests. The results are shown graphically on Figs. 15 to 20.

Transpiration.—One of the most important processes by which plants achieve their growth is that known as transpiration. The

structure of the leaves is such that their surfaces are covered with small openings or spores known as "stomata." Both surfaces are covered with these stomata, but the under surface has three or four times as many as the upper. Through these stomata the carbon from the air is admitted to the plant, and by a process in which sunlight forms an important part, this carbon, in combination with the water, is converted into starch, which forms a large part of the structure of many plants, particularly forage plants. The water taken in through the roots and flowing upward through the veins of the plant is evaporated from the surface of the leaves, serving to keep them moist and to operate the stomata, causing them to open or close and to regulate the quantity of air admitted, according to the requirements of the plant. This process has been described in various botanical and irrigation works,* and need not be mentioned herein. The quantity of water given off, however, is proportional to the quantity of starch produced by the plant and, consequently, to the growth of the plant itself, and the ratio of the quantity of water transpired to the quantity of plant matter produced, though not absolutely constant for certain plants, is within such close limits as to be to a certain extent measurable.

The quantity of transpiration, or the transpiration ratio, has been considered of such importance by scientists that many experiments have been made to determine it for many of the most common crops, particularly grains. Much valuable matter was contributed in this regard by the experiments of the University of Minnesota, prior to 1909, published by A. W. Sampson and L. M. Allen.† These studies showed the great effect of light on the quantity of transpiration, the lesser effect of heat, and the practically negligible effect of humidity. Fig. 8, reproduced from the "Minnesota Botanical Studies", shows the relations between the different phenomena mentioned.

Many other experiments have been made with soil pans in which plants are raised, the transpiration being measured by weighing the pan from time to time with its contents and weighing also the water applied, thus computing the water lost by transpiration. Descriptions of these experiments have been given in many scientific works and

* Such as King's "Irrigation and Drainage."

† "Minnesota Botanical Studies", Part 1, Vol. 4.

textbooks.* The determination in this manner, however, requires rather an elaborate experiment station and the constant attention of the observer.

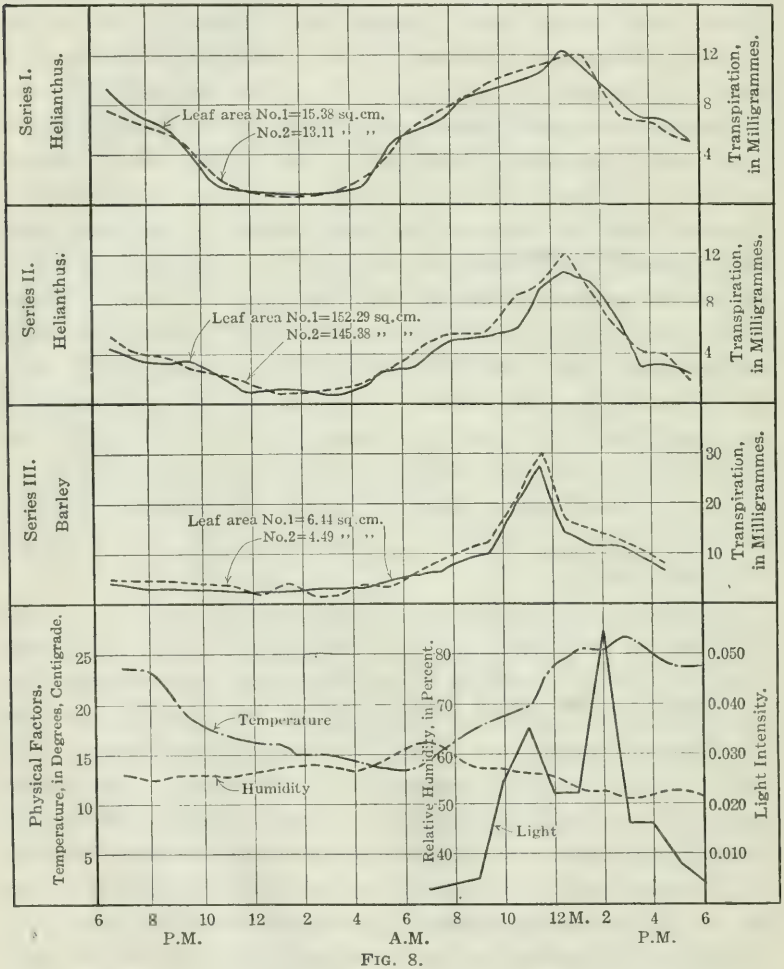

FIG. 8.

The transpiration ratio, or the relation of the quantity of water lost by transpiration to the quantity of dry matter produced for various crops, is shown in Table 10, taken from the work of Professor Fortier.

* Among these are "Irrigation and Drainage", by F. H. King; "Use of Water in Irrigation", by Samuel Fortier, M. Am. Soc. C. E.; and "Principles of Irrigation Practice", by John A. Widtsoe.

TABLE 10.—Transpiration for Various Standard Crops.

From "Use of Water in Irrigation", by Samuel Fortier.

Crop.	Location.	Experimenter.	Pounds of Water per Pound of Dry Matter.		
			Max.	Min.	Mean.
Wheat	Germany.	Sorauer.	708	....	708
	Germany.	Hellriegel.	390	328	339
	Germany.	Von Seelhorst.	333	....	333
	India.	Leather.	544	....	544
	Akron, Colo.	Briggs and Shantz.	534	468	507
	England.	Lawes.	235	....	235
	Logan, Utah.	Widtsoe.	489	427	458
	Davis, Cal.	Fortier and Beckett.	359	286	326
	Bozeman, Mont.	Fortier and Gieseker.	334	226	271
	Reno, Nev.	Fortier and Peterson.	395	309	360
Oats	Germany.	Wollny.	....	....	665
	Germany.	Sorauer.	....	....	600
	Germany.	Hellriegel.	464	339	401
	India.	Leather.	....	....	469
	Wisconsin.	King	526	502	514
	Akron, Colo.	Briggs and Shantz.	639	598	614
Barley	England.	Lawes.	262	258	260
	Germany.	Wollny.	....	....	774
	Germany.	Sorauer.	....	....	490
	Germany.	Hellriegel.	366	263	297
	Germany.	Von Seelhorst.	454	295	365
	India.	Leather.	....	....	468
	Wisconsin.	King.	401	375	388
	Akron, Colo.	Briggs and Shantz.	544	527	539
Corn	Germany.	Wollny.	....	....	233
	India.	Leather.	....	....	337
	Wisconsin.	King.	390	305	348
	Akron, Colo.	Briggs and Shantz.	420	319	369
Rye	Germany.	Hellriegel.	438	315	377
	Germany.	Von Seelhorst.	700	343	469
	Akron, Colo.	Briggs and Shantz.	....	....	724
Peas	England.	Lawes.	....	....	235
	Germany.	Wollny.	....	....	416
	Germany.	Hellriegel.	353	231	292
	India.	Leather.	....	....	563
	Wisconsin.	King.	....	....	477
	Akron, Colo.	Briggs and Shantz.	....	....	800
Potatoes	Germany.	Von Seelhorst.	294	268	281
	Wisconsin.	King.	....	....	423
	Akron, Colo.	Briggs and Shantz.	....	....	448
Alfalfa	Davis, Cal.	Fortier and Beckett.			
1st year			1 265	1 005	1 102
2d year			971	522	761
	State College, N. Mex.	.....	889	757	823
	Akron, Colo.	Briggs and Shantz.	....	....	1 068
Clover (red)	England.	Lawes.	....	....	251
	Germany.	Hellriegel.	363	297	330
	Wisconsin.	King.	564	398	481
Sugar beets	Logan, Utah.	Widtsoe.	....	....	497
	Akron, Colo.	Briggs and Shantz.	....	....	377
Rice	India.	Leather.	....	....	811

In describing experimental work done and deductions therefrom in connection with the water supply for Los Angeles, Cal.*, Mr. Charles H. Lee advances the theory that, if plants are cut, the water supply from their roots thus being instantly stopped, the transpiration will

* *Water Supply Paper No. 294*, U. S. Geological Survey.

continue from the stored water in the plants for a brief period at the same ratio as before they were cut. Acting on this theory, Mr. Lee made some experiments with alfalfa, using 15 min. as the period for which transpiration would remain constant.

In connection with the determination of the duty of water described herein, the writer made some experiments on the Chewaucan Marsh in order to determine the transpiration from sugar grass, following Mr. Lee's theory, and, though in doubt as to the maximum period for which transpiration will continue to be constant, he used 15 min. as the time, the same as Mr. Lee. It is interesting to note that, in experiments by pan methods, described later, the results are not

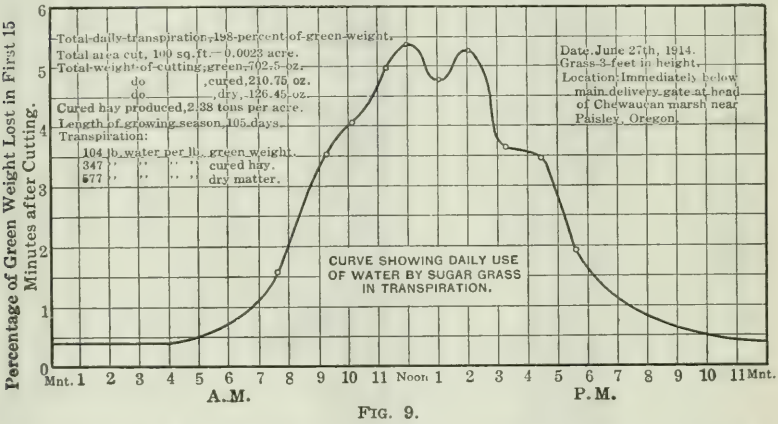
Percentage of Green Weight Lost in First 15 Minutes after Cutting.
Total daily transpiration, 198 percent of green weight.
Total area cut, 100 sq. ft.=0.0023 acre.
Total weight of cutting, green, 702.5 oz.
do cured, 210.75 oz.
do dry, 126.45 oz.
Cured hay produced, 2.38 tons per acre.
Length of growing season, 105 days.
Transpiration:
104 lb. water per lb. green weight.
347 " " " cured hay.
577 " " " dry matter.
Date, June 27th, 1914.
Grass, 3 feet in height.
Location: Immediately below main delivery gate at head of Chewaucan marsh near Paisley, Oregon.
CURVE SHOWING DAILY USE OF WATER BY SUGAR GRASS IN TRANSPIRATION.
Mnt. 1 2 3 4 5 6 7 8 9 10 11 Noon 1 2 3 4 5 6 7 8 9 10 11 Mnt.
A.M.
P.M.

FIG. 9.

incongruous as compared with the more accurate experiments made subsequently. Fig. 9 shows graphically the loss at various hours of the day, from the growing plants, in percentage of the green weight of the cut plants.

This experiment was performed by taking a typical growth of sugar grass of considerable extent, where the distribution of the plants was uniform, and laying this plat off in squares containing 10 sq. ft., each. From each of these squares the sugar grass was cut, and was weighed immediately on cutting and 15 min. later, the loss in weight for this time being computed as a proportion of the green weight at the time of cutting. Immediately after the first weighing, the plants were spread on a canvas and made to cover the same area as while standing; they were left in this position until the time of the second weighing. Fig. 6 shows the apparatus used for this

purpose, which consisted of a scale, weighing accurately to half ounces, up to a maximum of 4 lb.

The result of this experiment showed the total transpiration for the day as 198% of the green weight of the plant at that period of its growth. As the plant was then full grown, it was considered that the green weight would vary uniformly from the time that it sprouted to the time when it had attained its full growth, which would be from about March 15th to June 30th, or about 105 days. This would make the total transpiration during the growth of the plant 104 times its green weight at the time of cutting, or 104 lb. per pound of green weight at the time of cutting. Fig. 10 shows the loss of weight of the green cutting in curing for hay. It was found that after 3 days in ordinary weather the curing was complete, and the loss was approximately 70% of the green weight at the time of cutting.

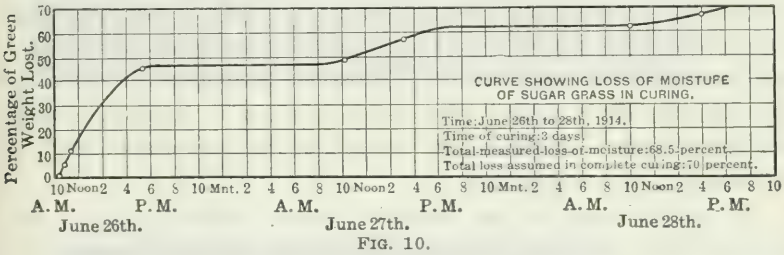
Percentage of Green Weight Lost.
70
60
50
40
30
20
10
0
CURVE SHOWING LOSS OF MOISTURE OF SUGAR GRASS IN CURING.
Time: June 26th to 28th, 1914.
Time of curing: 3 days.
Total measured loss of moisture: 68.5 percent.
Total loss assumed in complete curing: 70 percent.
10 Noon 2 4 6 8 10 Mnt. 2 4 6 8 10 Noon 2 4 6 8 10 Mnt. 2 4 6 8 10 Noon 2 4 6 8 10
A. M. P. M. A. M. P. M. A. M. P. M.
June 26th. June 27th. June 28th.

FIG. 10.

This would make the quantity of water transpired per pound of cured hay, 347 lb. It was also found that the cured hay contained about 40% of water; consequently, the transpiration per pound of dry matter would be 577 lb. This, however, is merely given for purposes of comparison with tables of transpiration for other plants which are reduced to a basis of dry matter produced, because, for the purpose of the experiments, the comparison was required with cured hay. Experiments were also made to ascertain whether or not the loss of weight was uniform for the first 15 min., and consequently equal to the transpiration rate. Although the experiments seemed to show that the rate of loss immediately after cutting was from 40 to 50% greater than after a period of 15 min., the chance of error in such small periods of time was considered great enough to warrant the experimenters in ignoring this result.

In the determination of the duty of water in Harney Valley, the experiments were more elaborate. As stated previously, the growth in

Harney Valley consists of various kinds of plants, and, consequently, experiments were conducted to show the transpiration for several kinds of growth, as follows:

Each experiment required an equipment consisting of three soil pans. Two of these were 35.7 in. in diameter and 4 ft. in depth, sunk 3 ft. into the soil, and having the excavated soil replaced in the pan in the same order as it was taken out, so that the top of the soil in the pan consisted of an unbroken sod containing the same growth as the remainder of the field. The third pan was 11.3 in. in diameter, 4 ft. in depth, sunk to the same depth as the other pans, and having the soil replaced therein in the same order, with the exception that all vegetation and plant growth were carefully removed, so that the pan would serve for soil evaporation only. As will be noted, the surface area of the two large pans was 1 000 sq. in. each, and that of the small pan was 100 sq. in. The evaporation pan was made of less diameter in order to maintain as nearly as possible the same conditions of shaded area as occurred in those pans covered with plant growth. Rain gauges were also maintained in connection with each set of pans, in order to take account of the precipitation during the period of the experiment.

At the beginning of the irrigation season, sufficient water was measured into each pan to bring the ground-water level therein to the same elevation as that in the soil of the field itself. Thereafter, observations were made at periods of from 4 to 10 days, and sufficient water was measured into each pan at each observation to keep the level at the same elevation as in the remainder of the field. A record was kept of the quantity of water introduced into each pan, the elevation of ground-water referred to the bottom of the pan, and such other data in regard to the growth of plants, the rainfall, and other conditions, as would aid in analyzing the results. At harvest time, sufficient water was introduced to bring the ground-water to the same elevation as that at the beginning of the test, the total water added thus equalling that used by the plants in transpiration plus the soil evaporation. In the final analysis of the duty of water, these two items were not segregated, but the total used in each pan was charged against transpiration and evaporation. For purposes of comparison with other transpiration data, however, the evaporation pan was used as a control, and the evaporation from

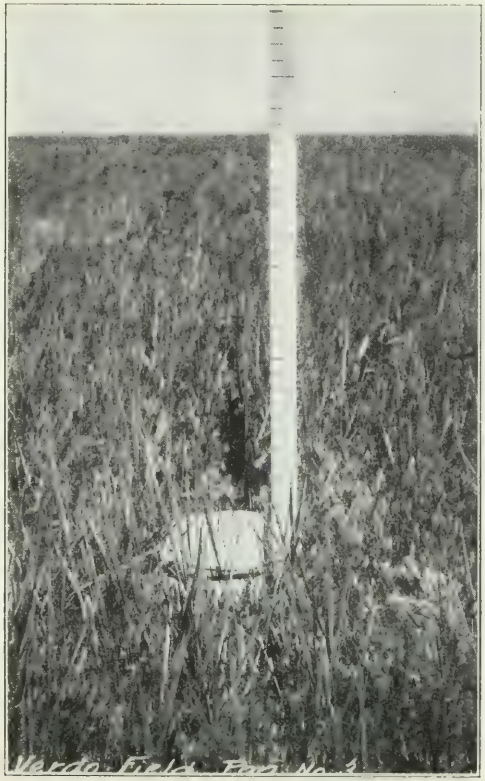

FIG. 11.—SOIL EVAPORATION PAN.

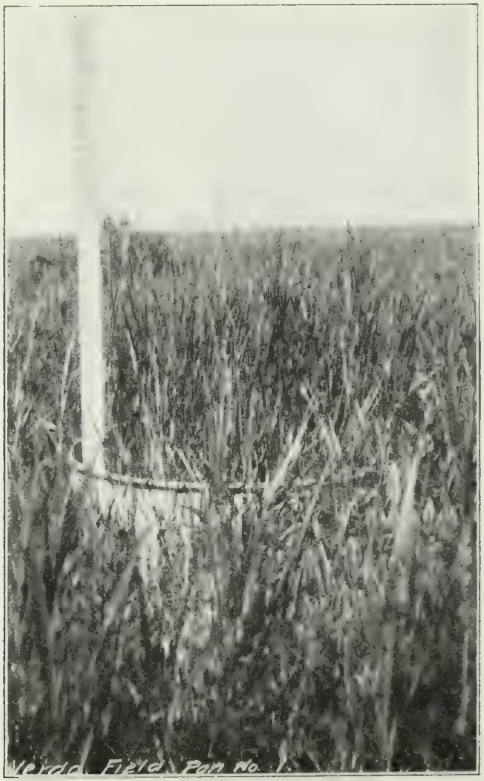

FIG. 12.—TRANSPIRATION PAN.

Fig. 13.—One of the Transpiration Pans After Harvesting the Crop.

Fig. 14.—Sinking Tube Through Sod by Turning.

it was deducted from the total in the other pans, in inches of height, the difference being considered as transpiration. It is not claimed that this will give absolutely accurate results as to transpiration, for the reason that conditions in the evaporation pan, on account of absence of mulch due to growing sod, will be somewhat different from those in the transpiration pans, but, as the area of the evaporation pan was made small enough so that the adjacent growth would protect the surface practically to the same extent as in the large pans, the results would be sufficiently close to give an approximation to the transpiration. Whatever inaccuracies might occur in the calculation of either evaporation or transpiration were eliminated by the use of the results from this pan as a combined figure for evapo-transpiration.

Figs. 15 to 20 show graphically the mass curve of the quantity of water added to each pan, exclusive of that added at first to bring ground-water to the starting elevation, and the fluctuation of ground-water in the pan during the period of the test. The tests covered the irrigation seasons of 1915 and 1916, and are interesting in showing the difference of conditions during the two seasons, there being a corresponding difference in evaporation, but a close agreement in transpiration ratio, as would be expected. The year 1915 was extremely dry, and the flood came down somewhat later than usual in the summer, which caused the length of the growing season to be very much less than its average, due to shortening both at the beginning and at the end. The season of 1916, however, was an extremely good one for irrigation, although the mean temperatures ran somewhat below the 11-year mean for which records are available.

In converting the transpiration ratio, as determined by these experiments, to the actual duty of water chargeable to transpiration, in acre-inches, the growth had to be estimated. Experiments were made in various localities as to the weight of the green cutting and the consequent hay produced; but, in order to use the data throughout the localities in question, average crop figures had to be utilized. Although the different experiments showed from 3 to 5 tons per acre of hay harvested from the Chewaucan Marsh, and from 2 to 5 tons from the Harney Valley fields, an inspection of crop reports showed that the average quantity produced from the former locality was slightly less than 3 tons per acre, and that harvested from the latter

locality was but little more than 1 ton per acre. Therefore, in converting the figures of transpiration to acre-inches, a crop intensity of $2\frac{1}{2}$ tons per acre was used for the Chewaucan Marsh, and $1\frac{1}{2}$ tons per acre for the Harney Valley.

EVAPORATION AND TRANSPIRATION TEST.
NEAR BURNS, ORE.
Location: Wickes Field. Mean of two pans.
Soil: 4 in. peat, 32 in. sediment.
Growth: Sugar grass and mixed meadow grasses.
Evaporation = 12.10 in.
Transpiration = 5.60 in.
Weight of cutting, green, 26.4 oz.
cured, 7.9 oz.
dry, 4.7 oz.
Transpiration ratio to green weight, 123
do hay " 409
do dry matter, 682

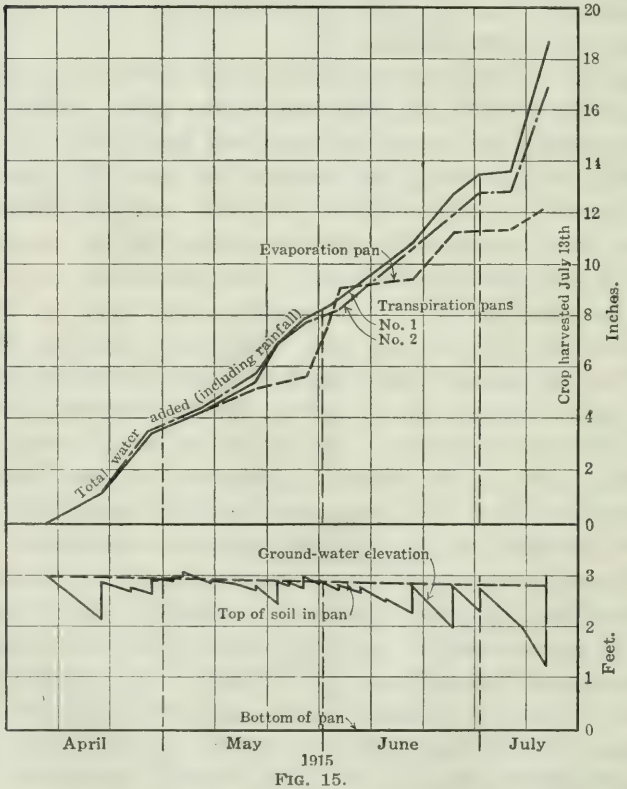

FIG. 15.

Soil Absorption.—When a quantity of water is applied to the surface of a dry soil, it sinks and is absorbed by the soil, traveling downward and laterally until the distribution is such that all is held by

capillarity. When a balance is obtained, the water again begins to travel upward to replace the moisture evaporated from the surface, and so continues until a condition is reached when the quantity of moisture is so slight that no travel will occur. Although the total voids may be equal, in two soils having particles the effective diameter of which is widely different, the storage capacity of the soil of finer texture is the greater, and likewise its resistance to percolation, due to the greater surface tension of the smaller particles. Different plants require different proportions of soil moisture in order to produce the best results. When the percentage of soil moisture drops below certain limits, the roots refuse to take up more, and the plant wilts. Table 11 gives the percentage of moisture below which the plant growth comes to a standstill and the plant wilts.

As the percentage of moisture required for plant growth exceeds the quantity which will be held by capillary action, it is clear that in permeable soils a balance cannot be reached, as previously stated, but there must be a continuous movement of water downward in irrigation. This is called percolation. If, however, the soil is underlain by an impermeable stratum which prevents a continuous downward motion, the soil moisture may be held above the wilting point without percolation losses.

EVAPORATION AND TRANSPIRATION TEST.
NEAR BURNS, ORE.
Location: Frye Field. Mean of two pans.
Soil: Silt loam
Growth: mixed meadow grasses.
Evaporation = 8.90 in.
Transpiration = 4.10 in.
Weight of cutting, green, 15.12 oz.
cured, 4.53 oz.
dry, 2.72 oz.
Transpiration ratio to green weight, 73
do hay " 242
do dry matter, 403

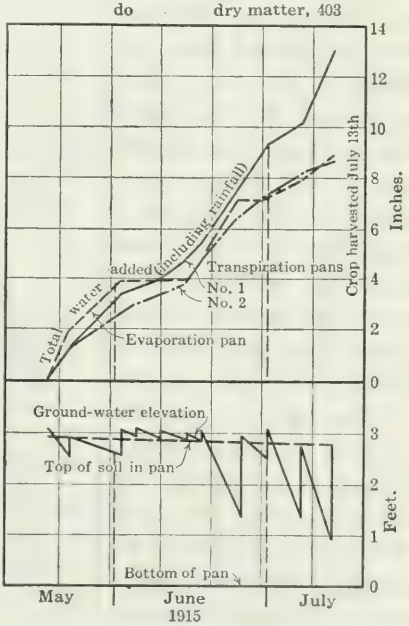

FIG. 16.

The foregoing description is given in order to show that soil absorption and percolation act in most conditions and for most crops; it is not given as a statement of the case for the experiments described in the following.

The soil of the Chewaucan Marsh consists of a layer of true peat, from 1 to 4 ft. thick, underlain by a few feet of mixed soil, under which occurs a bed of chalky soil of great depth. The chalky soil is continually saturated, and the ground-water during the non-irrigation season is within a few feet of the surface. During this season, as seen by Tables 5 and 6, an average of more than 8 in. of rain occurs, which offsets evaporation to such an extent as to prevent any great lowering of the ground-water level. As soon as irrigation is discontinued, the peat soil dries very quickly, but the lower stratum of the peat being very compact and the texture of the subsoil being very close, there is not a great deal of water drawn from the subsoil. Post holes dug during October showed a low percentage of moisture at a depth of 3 ft.

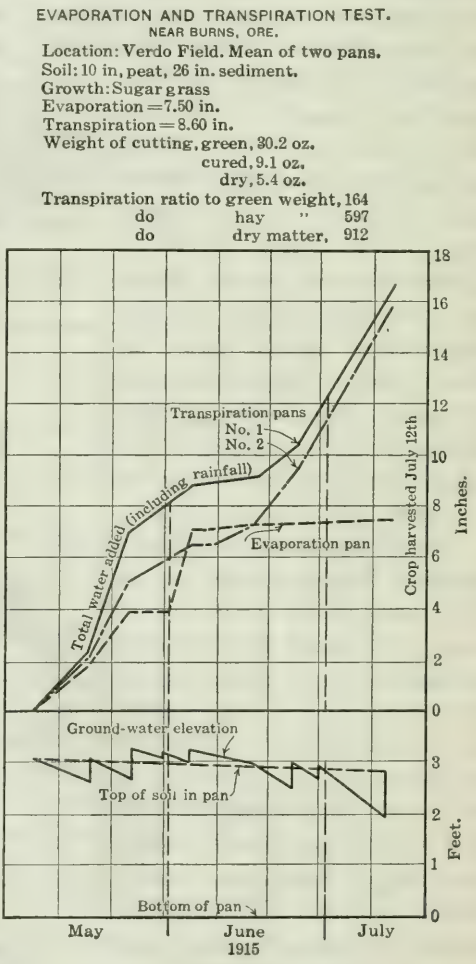

FIG. 17.

The character of the crops on Chewaucan Marsh is such as to require continuous flooding to a depth of several inches above the surface during the irrigation season and the continuous movement of the water during that time. An examination of the nature of the growth on different parts of the marsh, with an analysis of the topographical conditions, shows that any departure from this condition of continuous flooding has an immediate effect on the character of the growth. Wherever the flow of water is checked,

EVAPORATION AND TRANSPIRATION TEST.
NEAR BURNS, ORE.

Pan No. 2, Wickes Field.
Soil: 4 in. peat, 32 in. sediment.
Growth: Sugar grass and mixed meadow grasses.
Evaporation = 15.78 in.
Transpiration = 8.70 in.
Weight of cutting, green, 41 oz.
cured, 12.5 oz.
dry, 7.5 oz.
Transpiration ratio to green weight, 122
do hay " 402
do dry matter, 669

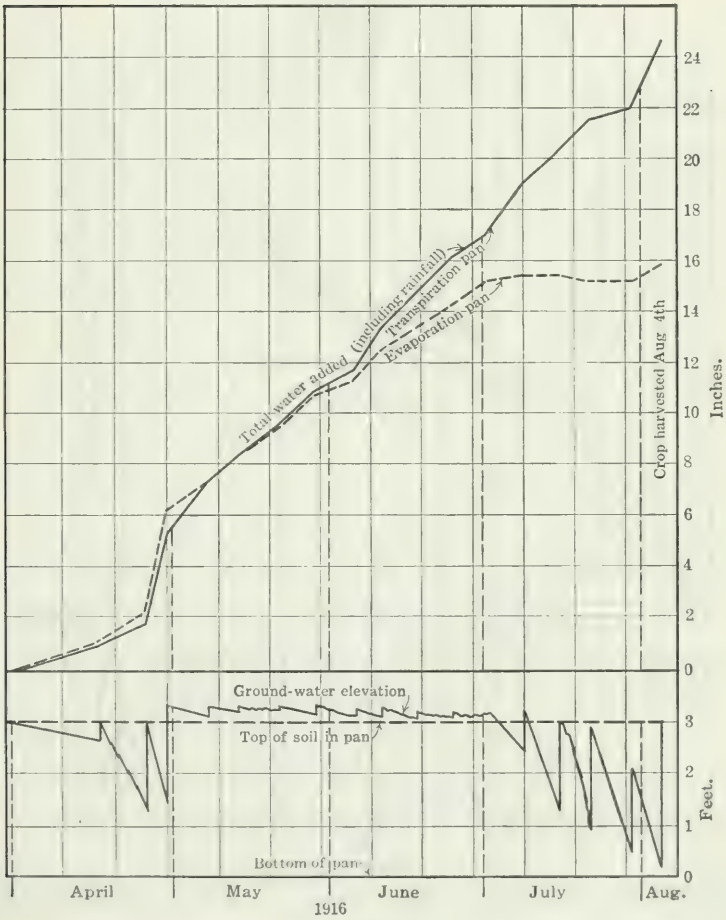

FIG. 18.

so that it is practically at a standstill, the beneficial growth is changed to tules, cat-tails, and other plants having no crop value. Wherever knolls occur, so that the water does not cover the surface, the growth

EVAPORATION AND TRANSPIRATION TEST.
NEAR BURNS, ORE.
Location: Frye Field. Average of two pans.
Soil: Silt loam.
Growth: mixed meadow grasses.
Evaporation = 15.62 in.
Transpiration = 3.55 in.
Weight of cutting, green, 28.62 oz.
cured, 8.59 oz.
dry, 5.15 oz.
Transpiration ratio to green weight, 71
do hay " 238
do dry matter, 398

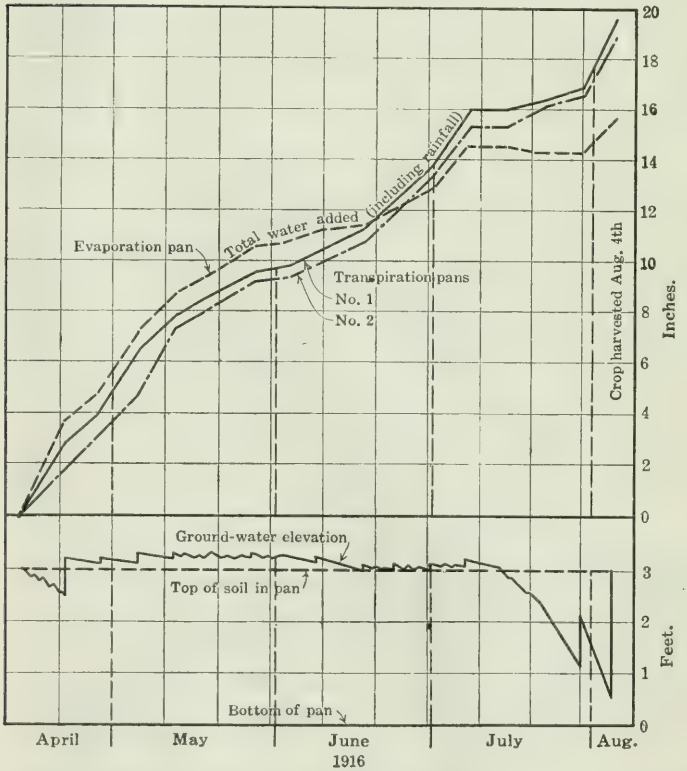

FIG. 19.

changes to fox-tail, bluejoint, and other grasses of less value than those covering the larger part of the marsh.

It is certain, therefore, that no other system of irrigation than that described would be practicable. In reference to the quantity

EVAPORATION AND TRANSPIRATION TEST.
NEAR BURNS, ORE.

Location: Verdo Field. Average of two pans.
Soil: 10 in. peat, 26 in. sediment.
Growth: Sugar grass.
Evaporation = 17.33 in.
Transpiration = 4.95 in.
Weight of cutting, green, 27.5 oz.
cured, 8.25 oz.
dry, 4.95 oz.
Transpiration ratio to green weight, 104
do hay " , 346
do dry matter, 577

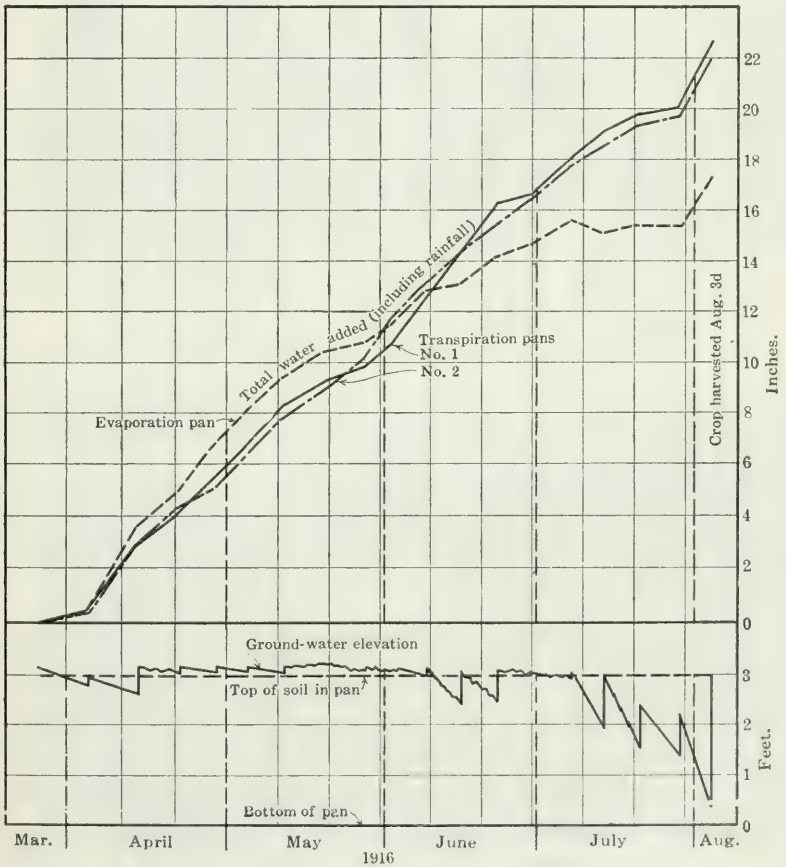

FIG. 20.

TABLE 11.—Relation Between Hygroscopic Moisture Content and Moisture Content at Which Plants Begin to Wilt.

Bulletin 50, U. S. Bureau of Soils.

Soil.	Water content at saturation. Percentage.	Hygroscopic water content. Percentage.	Water content at which plants begin to wilt. Percentage.
Coarse sand	26.5	0.42	1.5
Medium fertile garden soil	43.9	1.68	4.6
Infertile sandy muck	41.4	0.97	6.2
Sandy loam	43.3	2.40	7.8
Very fertile calcareous soil	38.3	3.65	9.8
Peat soil	274.0	20.6	49.7

Percentages given by weight, and not by volume.

of water absorbed by the soil, therefore, one must consider, not only a thoroughly saturated soil, but a water cover of sufficient depth to insure a surface velocity sufficient to supply the needs of the beneficial grasses.

To determine the quantity of water which would be absorbed by the soil at the beginning of the irrigation season, the following experiments were performed, and the following calculations made:

First.—The porosity of the soil was determined, roughly, by making an excavation to a depth of 4 ft., and accurately measuring its size. A water-tight box was then constructed having the same cubical capacity as the excavation. After drying the earth taken from the excavation, it was replaced in the box in such a manner as to occupy the same space as it did in place. Water was then measured into the box until the soil was thoroughly saturated, and thus the percentage of voids was determined, and found to be 58 per cent.

Second.—From the writer's previous work in soils of a similar nature, it was found that, with ground-water at a depth of about 8 ft., soil evaporation would be negligible, and, for any depth to ground-water of less than 8 ft., the quantity of evaporation would be proportional to the depth. A similar conclusion was reached by Mr. Lee, in his work previously cited.* With this as a basis, and using the mean monthly temperatures and mean monthly precipitation for the non-irrigation season, namely, from July 1st to March 15th, Table 12, showing the depths to ground-water at different periods,

* *Water Supply Paper No. 294*, U. S. Geological Survey.

FIG. 21.—SINKING TUBE IN SUBSOIL BY DRIVING.

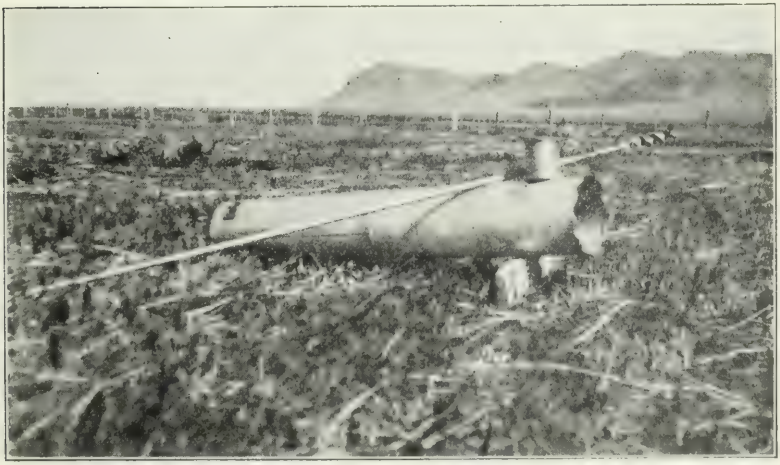

FIG. 22.—APPARATUS USED IN SOIL ABSORPTION TEST.

was computed. In doing this, it was assumed that irrigation would be stopped on July 1st and that the marsh would be uncovered progressively from July 1st to 31st, inclusive—evaporation thus taking place over half the marsh during July.

TABLE 12.—Progressive Drying of Soil During Non-Irrigation Season for Chewaucan Marsh.

Month.	Evaporation from water surface, in inches.	Per-centage.	Sub-surface evaporation, in inches.	Rainfall, in inches.	Depth to ground-water, in inches.
July	4	97	3.88	0.80	5.50
August	7	90	6.30	0.37	15.35
September	5	82	4.10	0.76	20.85
October	3	77	2.31	0.82	23.33
November	1	76	0.76	1.32	22.63
December	1	76	0.76	0.96	22.38
January	1	77	0.77	1.51	21.26
February	1	78	0.78	1.21	21.03
March 1st to 15th	1	78	0.78	0.52	20.66
	24	.....	20.44	8.27	.....

From Table 12 it is seen that ground-water would reach its lowest depth on October 31st, and that, on March 15th, it would be at a depth of 20.66 in. below ground surface. It is not meant by this that the actual ground-water would be at this depth, but that the condition of the ground for the first 8 ft. would be as though ground-water was at this elevation and the soil was dry above it. The quantity of subsoil evaporation to be replaced at the end of this period is seen from Table 12 to be 20.44 in., less a rainfall of 8.27 in., during the same period, or a net depth of 12.17 in.

As a check on this computation, and also in order to determine the rate of absorption, a special apparatus was devised for making a more accurate test in place. This apparatus (Figs. 14, 21, and 22) consisted of a heavy sheet-iron tube, 5 ft. long and 10 in. in diameter, to the top of which was riveted a heavy band for driving. The bottom of the tube was sharpened to a fluted knife-edge for the purpose of cutting through the sod into the subsoil before beginning to drive. This tube was set up vertically over the place to be tested, and turned with a long lever until it had cut through the sod to a depth of about 12 in. It was then driven with wooden mauls through the soil and into the chalky layer below so as to enclose an area of the original

soil 10 in. in diameter in the water-tight tube. Borings, adjacent to the tube, were made with a soil auger in order that the nature and depth of the soil could be recorded. Water was then measured into the tube with a quart measure, and replenished at stated intervals, the depth of water added being recorded at the time. The top of the tube was kept covered in such a manner that no evaporation could take place. The result of this test is shown on Fig. 23.

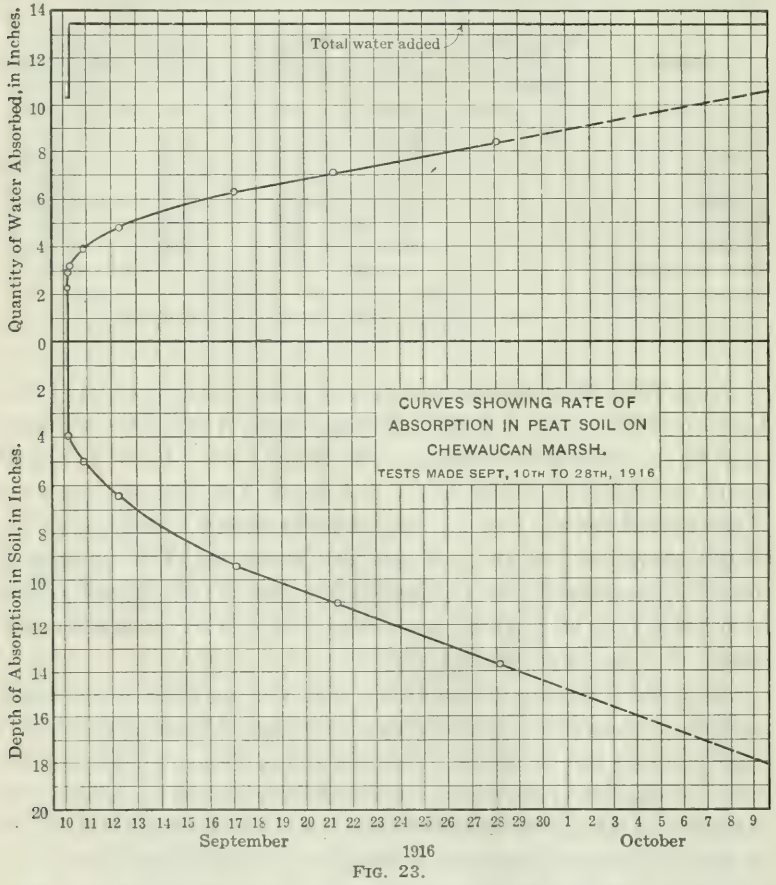
Quantity of Water Absorbed, in Inches.
Depth of Absorption in Soil, in Inches.
Total water added
CURVES SHOWING RATE OF ABSORPTION IN PEAT SOIL ON CHEWAUCAN MARSH.
TESTS MADE SEPT, 10TH TO 28TH, 1916
September
October
1916

FIG. 23.

It will be noted from the curves on Fig. 19 that the water sinks almost immediately to a depth of 4 in., the rate of absorption then gradually diminishing until a depth of about 10 in. is reached, when the rate of absorption becomes fairly uniform. It will also be noted that the soil requires about 6 weeks to reach a saturated condition.

It is probable that the actual rate of absorption on the marsh is somewhat greater than that shown in the test. During dry weather, the surface of the marsh cracks to some extent, the cracks extending some distance into the soil. These cracks lead the water to a good depth immediately, and allow it to distribute laterally as well as vertically. In the test, however, no crack was included within the limits of the cylinder sunk into the soil, and, therefore, this lateral distribution would be eliminated.

In Harney Valley, the soil in general is of a different nature. The writer found, by borings made in October, 1916, about 2 months after harvest time, that the soil was quite dry for a depth of 2 or 3 ft., gradually increasing in moisture below that depth until a saturated condition was reached at a depth of about 5 ft. Soil samples were taken at various locations, and a porosity of from 27 to 52% was found. The subsoil is such that considerable percolation occurs, both vertically and laterally, the latter being very noticeable in the growth of grasses and the deposit of alkali on areas not irrigated. It was found that lateral percolation would be apparent at a distance of 500 ft. from the nearest place where water was applied on the surface.

As the rainfall from October to March is generally sufficient to offset soil evaporation, and as the soil in test holes was found to be dry for a depth of 3 ft., the quantity of moisture which would be absorbed in the 3 ft. would vary, according to the texture of the soil, from 10 to 18 in., or an average of about 14 in.

Percolation and Run-Off.—One of the principal losses of water in irrigation, and likewise the hardest component to determine, is that of percolation and run-off. As heretofore stated, it is impracticable to supply to the soil the exact quantity of moisture needed for plant use and evaporation and still keep it in the right condition of moisture to obtain the best growth. The best yields per acre are obtained in soils which have a natural drainage, that is, where the soil itself is underlain by a more open subsoil, permitting the excess waters to percolate freely and be carried to some watercourse. Where underground drainage is not obtained in this manner, it sometimes happens that the excess water applied in irrigation accumulates in the soil and gradually raises the water-table until harm is done to the plant life. In these cases, it becomes necessary to put in an

artificial system of drainage, consisting either of open-drain ditches or buried tile drains leading to some watercourse where the excess water may be discharged.

In the case of the growths considered in this paper, however, no harm is done by lack of under-drainage; on the contrary, the plants thrive in a saturated soil, and, in the case of sugar grass, even require moving water on the surface of the ground It is a decided benefit, therefore, to have an impermeable subsoil and a lack of under-drainage, as this eliminates losses by percolation which would otherwise occur.

On the Chewaucan Marsh, as heretofore stated, the subsoil is of an impermeable nature, and there is practically no loss from percolation, but, as the sugar grass requires a saturated soil and free water moving over its surface, and as the configuration of the marsh is such that an absolute uniform depth cannot be obtained, the depth must be such as to cover the high points. The least mean depth which will cover the marsh and submerge it as a whole was found to be from 4 to 6 in., at which the depth would be practically zero at the high points.

The least velocity at which the surface water needs to move, in order to prevent it from becoming stagnant and changing the character of the growth, has never been determined. A determination was made of the velocity under present conditions, both by calculation from physical conditions and by observation of the current as it exists, and this velocity was used to compute the quantity necessary for run-off. It is possible that works might be established by which this velocity could be made less without harm to the plants, but this is not known, and it is very doubtful if the velocity could be diminished much without producing damaging conditions.

Using the velocity thus obtained and the cross-section of the water cover at various points on the marsh, the writer calculated that the loss of water by surface run-off was about 23 in. in depth over the acreage irrigated.

As before stated, the nature of the soil, the growth, and the topography of the country in the Harney Valley are much different from that on the Chewaucan Marsh. The surface soil in Harney Valley consists of a medium silt loam on the surface, with occasional areas of peat soil, the whole being underlain with a sedimentary silt having particles with an effective diameter of about 0.5 mm. The valley is cut up considerably by watercourses, dams in the latter serving to

deliver the water to the land, the excess being dispersed by run-off, by longitudinal percolation to Malheur Lake, and by lateral percolation to the streams below the dams, and in these it continues to the lake.

Excepting for the sugar-grass areas, continuous surface flooding would not be necessary, from the standpoint of growth; but, considering the fact that both soil and water are quite alkaline, such flooding is necessary to keep this alkalinity from becoming detrimental to growth. Outside of sugar grass, the growths found in the valley thrive about equally under continuous flooding and intermittent flooding sufficient to keep the soil saturated. However, as the latter is strongly impregnated with alkaline salts, any condition which will cause the soil water to be brought to the surface and evaporated, will cause a deposit of the salts, which is destructive to vegetation. By continuous flooding, the salts are prevented from being deposited, and the growth continues until the final shutting off of the water, when they are deposited up to the time when the soil is dry, but too late to damage the growth to any extent. The salts deposited after the irrigation season are washed off with the first rush of water in the spring, and this allows the new growth to start.

The loss by longitudinal percolation is slight. As the effective diameter of the soil particles is about 0.5 mm., and the surface slope toward the lake is about 0.08%, the rate of percolation, longitudinally, would be only about 9 in. per day; but, as the valley is traversed by deep watercourses to which the water may percolate and be carried off directly, there are slopes as great as 4 or 5%, which would cause velocities of percolating water as great as 50 ft. per day. Even at this rate, however, percolation is small, as compared with surface run-off, and as there is great likelihood of error in calculating either one, it was considered in this investigation as included in the run-off.

Conditions of run-off were much the same as on Chewaucan Marsh, except that the water was not collected into a single channel on its discharge from the irrigated area, as in the case of the Chewaucan Marsh, and, therefore, no check could be made on the calculations as to the quantity of run-off. The country was also much more irregular than the surface of the marsh, and thus many knolls were left without surface cover, and, on the areas affected, the depth had to be greater than in the case of the marsh, in order to cover as much ground as possible. For this reason, the surface velocity of the irrigating streams

was greater than on the Chewaucan Marsh, although the surface slope was somewhat less. It is safe to provide a run-off of at least 40% of the total delivery in order to protect against alkali. This would give about 24 in. of depth over the irrigated area.

Conclusion.—Though the investigation herein described was made necessary by the system of irrigation and a class of growth about which little was known, the same method of analysis may be used on any class of irrigation and any growth. By a study of the plant and its needs in regard to soil moisture, the method of application of the water may be determined; and by a study of soil and climatic conditions, an analysis of the use of this water may be made, which will vary in accuracy according to the extent of the investigation. It is not claimed by the writer that this method is always superior to that of direct trial; nevertheless, it is useful in some cases where that is impracticable, and in other cases it will give results which, taken in conjunction with the duty obtained by direct trial, will aid in the elimination of losses, the improvement in yield, and the betterment of irrigation methods.

In conclusion, the author wishes to acknowledge the assistance of Carl J. Rhodin, M. Am. Soc. C. E., to whom is largely due the work of comparison of evaporation data; and of Professor P. B. Kennedy, of the University of California, whose classification of the different plant growths encountered and study of the reasons for their occurrence were invaluable.

The writer has purposely omitted many of the factors which aided him in arriving at his conclusions, and even much of the detail in regard to the conclusions themselves, as the investigations described herein were for use in certain litigation which has not yet been brought entirely to a close. The purpose of this paper, therefore, is merely to show the methods used, and any figures given will be considered as merely exemplifying such methods and not as final determinations of results.

DISCUSSION

Mr. Versluys.

J. C. Van Reigersberg Versluys,* Esq.—The subject treated in this paper is of great interest to Java, the country from which the speaker has just come.

Java, one of the larger islands of the Netherlands East Indies, has a population of about 38 000 000, and the people live almost entirely on rice. The density of population is very great as the island is only about one-third as large as California. The birth rate is very high, and it must be expected that the population will be doubled within the next 40 years.

The area planted with rice amounts to about 4 000 000 acres, of which 2 500 000 acres are irrigated. The average yield of irrigated fields is about 1 000 lb. of eatable rice per acre. All rice is grown on small farms, most of them of from 1 to 2 acres. The rice farmer is a light-hearted, easily satisfied man, who does not work hard, but brings up his family on $60 a year. Of course, this cannot go on, for as civilization advances, the cost of living will continue to rise—the American moving pictures, for instance, have become a necessity in the larger cities.

As almost all the virgin soil is now in use, the Netherlands Government has been induced to undertake extensive investigations with reference to the possibilities of increasing the rice crop in accordance with the needs of the people. Of course, as the best and most easily irrigated holdings were taken up long ago, the main proposition now is to make the best use of what is still left, and improve the existing systems. The average yield of non-irrigated farm lands is from 50 to 55% of those which are irrigated.

Artificial fertilizers have been tested; they increase the output considerably, but have proved too expensive. The natural fertilizers are in the soil itself, which is generally volcanic. Irrigation helps to supply nitrogen, but phosphorus is the most important element in rice.

Many questions dealt with by Mr. Hammatt are also of the greatest importance in Java. There is not enough water to provide all the later irrigation systems with the conservative quantity of from 20 to 30 in. per year. The latest conclusion arrived at is that the duty of the water cannot be dealt with separately from local geological conditions.

The speaker regrets very much not being able to go further into details, but he has no notes with him to which he can refer.

Mr. Rockwell.

W. L. Rockwell,† M. Am. Soc. C. E. (by letter).—In the field of irrigation engineering the economic duty of water, a subject regarding which comparatively little is known, is of prime importance. In Texas

* Member of the Royal Institute of Engineers, The Hague, The Netherlands.

† San Antonio, Tex.

Mr. Rockwell. this study has been progressing for more than 3 years, and has included the application of water to cultivated and uncultivated fields, with and without crops, and daily sampling at intervals of from 5 to 30 days, to depths of from 6 to 12 in., both before and after irrigation. During the same periods, an investigation of the evaporation from water and soil surfaces has been conducted. The many and various details are made as comprehensive, and the work is done as accurately, as conditions will permit.

Mr. Stevens. J. C. Stevens,* M. Am. Soc. C. E. (by letter).—The author defines the duty of water, as used in his paper, to be "the quantity which can be applied with the greatest efficiency, under given conditions, for the production of crops."

It appears to the writer that the author's study merely resolves the quantity of water applied in practice into its component parts, without touching on the matter of efficiency at all.

Doubtless, the author concludes that the duty given is the efficient one, the data forming the basis of his conclusion being withheld for the very excellent reasons given in his closing paragraph.

The total quantity necessary for the crops in question, and its component parts, in the terms used by the author, are given in Table 13. To these quantities must be added the losses incident to the transportation of the water to the point of use, and storage losses, if any. Hence, the duty given in Table 13 appears to be very low indeed.

TABLE 13.

Component.	Harney Valley.	Chewaucan Marsh.
(1) Soil Moisture	11.0 inches.	12.2 inches.
(2) Soil Evaporation	10.8 "	10.5 "
(3) Plant Use	8.0 "	14.5 "
(4) Soil Losses	24.0 "	23.0 "
Totals	56.8 inches.	60.2 inches.
Acre-feet per acre	4.73	5.00

Good yields of alfalfa, wherever the climatic conditions favor its growth, can usually be secured with about half the foregoing quantities, net, on the land. Moreover, alfalfa is generally conceded to require more water than any other general crop growth in the United States.

The questions naturally arise: Is the land thus situated, where the total water supplies are limited, being put to its highest use? Would it not be far better to put twice the area under cultivation with the same water supply?

* Portland, Ore.

Mr. Stevens.

It is seen that the largest single component of the water duty is "Soil Losses". This is treated under the caption "Percolation and Run-off," from which it appears that these particular grasses require flowing water during the growing period, in order to produce aeration and carry off the alkaline salts.

The author states that, in Harney Valley, except for the sugar grass areas, continuous surface flooding is required in order to avoid alkalinity, and not for plant growth. In other localities the accepted method of removing injurious alkaline salts is by drainage. If open ditch drainage will obviate the necessity of continuous flooding, is not that the first point of attack, in order to determine the efficient use of water in this locality?

The writer knows the two areas in question only by reputation and cursory examination. He knows that in both valleys there are large areas of land and small quantities of water, and it would seem that the only justifiable use of the limited water supply is one that will put the greatest number of acres under cultivation and create the greatest number of homes.

The writer awaits with interest the remaining data which the author undoubtedly has obtained. For example, it would be interesting, indeed, to know the total quantity of water consumed in the two valleys during the two seasons in question. On Chewaucan Marsh this could have been, and perhaps was, determined with ease by measuring the total inflow and outflow on the upper and lower marshes at the three constricted portions of the valley. This consumption should be comparable with the sum of the first three component parts of the water duty previously given, and, in the writer's belief, will be considerably less than the 37.2 in. which the author's analysis seems to indicate as the required quantity.

In Harney Valley the problem of determining the consumption is not so simple, but, doubtless, natural facilities were found where it could be determined for certain representative areas.

The writer is delighted with this paper. Mr. Hammatt has attacked an old problem in a novel manner, and has arrived at results which contain an element of surprise. It is to be hoped that, when the time is propitious, he will give the Engineering Profession the benefit of all the data he has gathered.

The foregoing discussion was written after a careful reading of the paper, but without analyzing the processes by which the author reached his conclusions. The writer, however, has since had occasion to inquire more deeply into the subject, and has also secured additional data, relative to the use of water on Chewaucan Marsh, which throw some extremely interesting side-lights on Mr. Hammatt's conclusions.

Mr. Stevens.

As these data are matters of public record, and appear in publications of the U. S. Geological Survey, the writer feels that there can be no breach of ethics in using them.

In Table 13 the writer gave the component parts of the duty as fixed by Mr. Hammatt. These component parts will be considered in the order in which they are treated by the author.

Evaporation.—The writer begs to call the author's attention to the results of a 6-year record of evaporation from a standard U. S. Geological Survey floating pan in the Klamath River, at Keno, Ore., 1904-1909.* This record was not used by the author.

After making some minor adjustments for missing winter records, the 6-year average evaporation and average monthly temperature given in Table 14 has been used by the writer.

TABLE 14.—Monthly Evaporation and Temperature Record at Keno, Oregon.

Average of 6 Years, 1904-09; Elevation, 4 090 Ft.

Month.	Evaporation at Keno.	Air Temperature at Klamath Falls.
January	0.50	32.0
February	0.60	35.6
March	1.09	38.0
April	3.13	46.4
May	4.14	49.9
June	5.16	57.4
July	6.44	69.2
August	6.46	66.1
September	4.65	59.2
October	2.26	50.0
November	1.07	40.0
December	0.80	30.8
The year	36.30	47.9

This record for 1909 is simultaneous with that at Ady used by the author; yet the evaporation at Keno was considerably less than that recorded at Ady. Moreover, the evaporation for the Ady station, as given by the author in Table 8, is not the Ady record.

The record for the Ady station for 1909 is given in "Working Data for Irrigation Engineers", by Moritz, whom the author quotes as his authority. Moritz takes his figures from Professor Bigelow's Abstract of Data No. 4 entitled "A Provisional Statement Regarding the Total Amount of Evaporation by Months at 23 Stations in the United States, 1909-10."

* *Water Supply Papers Nos. 177, 213, 251,* and *271.* U. S. Geological Survey.

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Table 15 is a comparison of the figures as given by Professor Bigelow and Mr. Moritz with those used by the author; also a comparison of the Ady and Keno records for 1909, with corresponding temperatures.

It appears from Table 15 that the evaporation from the Ady pan was 15.5 in. more than from the Keno pan. The reasons are apparent when the physical characteristics of the two exposures are compared.

Keno Pan.—3 ft. square, 18 in. deep, floating in Klamath River, a stream about 400 ft. wide; mean depth, 8 to 10 ft.; discharge, 1 000 to 3 000 sec-ft.

Ady Pan.—4 ft. diameter, 10 in. deep, floating in stagnant shallow water of a borrow-pit alongside of railroad embankment.

The water temperatures of the borrow-pit were much higher than those in Klamath River, which chiefly accounts for the increase in evaporation.

TABLE 15.—Comparison of Evaporation Records at Ady and Keno, Oregon, 1909.

Month.	Ady.		Keno.			
				Mean Temperature.		
	Hammatt.	Bigelow (also Moritz).	Evaporation.	Water in pan.	Water in river.	Air at Klamath Falls.
January	0.50	0.50*	0.02	36	36	32.2
February	1.25	1.25*	0.18	39	38	32.5
March	3.57	3.57	0.29	44	41	38.4
April	4.64	6.64	3.87	59	54	43.8
May	6.48	7.15	4.47	61	57	49.0
June	7.78	6.99	6.65	68	68	58.2
July	7.65	8.01	6.06	68	66	61.2
August	7.64	9.21	6.65	74	72	64.8
September	5.26	6.13	1.68	62	61	59.3
October	2.87	2.50	2.76	54	54	48.9
November	1.00	1.00	0.83	43	43	37.5
December	0.50	0.50*	.....	.....	.....	27.0
The year	49.14	53.45	38.0*	.....	.....	46.1

* Estimated.

The results of experiments on evaporation at the Denver Laboratories by Mr. R. B. Sleight,* show that:

1.—Evaporation from flowing water is 8% greater than from still water, all other factors being equal.

* *Journal of Agricultural Research*, July 30th, 1917.

Mr. Stevens.

2.—An increase in water temperature produces corresponding increases in evaporation, as follows:

2° increase in temperature produces 10% increase in evaporation.

4° increase in temperature produces 19% increase in evaporation.

6° increase in temperature produces 30% increase in evaporation.

10° increase in temperature produces 55% increase in evaporation.

16° increase in temperature produces 100% increase in evaporation.

The increase at Ady is 41%, which would be produced by a mean water temperature increase only 8° greater than that at Keno. The Ady record, however, was not used by the author in deducing the evaporation for Chewaucan Marsh.

It has been shown that the correct figures for evaporation at Ady are already 40% too high, yet the author increases them by 7½% for differences in elevation, and then calls attention to the fact that they fit his figures deduced from Owens River, California.

Now examine the author's deduced evaporation for Chewaucan Marsh, in which a 4-year record from Owens River near Independence, Cal., was used. He first increases the evaporation at Independence by 16% for a corresponding difference in barometric pressure.

The author states "that evaporation at different elevations is in inverse proportion to the barometric pressure." He then gives the pressures at Independence and Chewaucan as 26.0 and 25.35 in., corresponding to elevations of 3 775 and 4 410 ft., respectively. Here is the equation:

$$\text{Evaporation at Elevation } 4\,410 = \text{Evaporation at } 3\,775 \times \frac{26.00}{25.35}.$$

This is an increase of 2½%, and not 16%, as stated by the author. The increase in elevation is 16.8%, which is probably what he had in mind.

In the paper cited by the author, "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico," by Messrs. Duryea and Haehl*, Subsidiary Conclusion (*e*) states their conclusions as to the relation between elevation, mean temperature, and evaporation, thus:

"(*e*)—In the Great Plateau Region, and at elevations above sea level of 600 to 5 000 ft. and monthly mean temperatures higher than 70° Fahr., the increases in the evaporation depth at any given mean temperature are directly proportional to increases in elevation; but at monthly mean temperatures lower than 70° Fahr., and at the lower elevations, the evaporation depth at any given mean temperature increases with the increase in elevation, but less rapidly than in direct proportion."

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1829.

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The writer positively denies that any such relation as that just quoted, or as modified by the author, has been established in the paper cited or elsewhere. A perusal of the paper in question and of the discussions which follow it will convince the most credulous that no such relation exists. As a matter of fact, such information as we have on the subject indicates the opposite tendency, namely, a decrease in evaporation with an increase in altitude.

An article by Samuel Fortier, M. Am. Soc. C. E.,* gives the results of experiments on Mt. Whitney, Cal., to determine the influence of altitude on evaporation. Evaporation records were obtained at stations varying in elevation from 4 500 to 14 500 ft.

Briefly stated, the results of this investigation indicated a consistent and steady decrease in the evaporation from the lower to the higher elevations, that at 10 000 ft. being 40% less than that at 4 500 ft. Professor Fortier concludes:

"While these [non-uniformity of exposure] doubtless influenced the results to some extent it is safe to conclude that altitude was the controlling factor."

It is the writer's belief, however, that the controlling influence, in the Mt. Whitney experiments and elsewhere, is water temperature, and that it is just as easy and just as logical to attempt to establish a relation between the water temperatures of Owens River and Chewaucan Marsh and their respective altitudes, as it is between the evaporation and elevation of the two places.

Moreover, if the evaporation varies inversely as the barometric pressure, it would be infinitely great in a vacuum, which is an absurdity.

After correcting the Owens Valley records for elevation, the author proceeds to correct them for mean monthly temperatures. This process can only be defended on the theory that air temperatures are some definite index of water temperatures. That there is a broad general relation between air and water temperatures is admitted, but when the great complexity of conditions, such as origin of water (*i. e.*, springs or surface), exposure, depth, currents, atmospheric conditions, etc., are considered, the relation must necessarily be so broad and general that the author's use of it cannot be defended on any logical basis.

By way of illustration, it is interesting to note the effect on evaporation and plant transpiration of the volcanic eruption of Mt. Katmai, Alaska, during the early part of June, 1912. This effect was noted by Messrs. Briggs and Shantz during their experiments to determine the water requirements of plants at Akron, Colo.†

These gentlemen increased the water consumption of plants during 1912, as observed by them, 30% in determining mean values from the

* "Evaporation Losses in Irrigation," *Engineering News*, September 19th, 1907.

† "The Water Requirement of Plants," *Journal of Agricultural Research*, October 15th, 1914, by Lyman J. Briggs and H. L. Shantz.

Mr. Stevens.

3-year record, 1911, 1912, and 1913. They also show that evaporation from a free water surface during June, July, and August, 1912, was 22% lower than the average of 1911 and 1913. These reductions in both evaporation and plant transpiration were charged directly, by them, to the volcanic disturbance cited.

When such remote causes are seen to exert such powerful influences, it would seem that the results obtained by the author are absurd. He increases the Owens Valley records 16% for elevation (our present knowledge indicates they should be reduced, if corrected at all); next, he reduces them for temperature (with no basis of fact); then he increases them 33% for shallow water (an utterly unfounded assumption); finally, he decreases them by varying quantities for plant growth (purely guesswork); and he confidently states that the mean evaporation on Chewaucan Marsh, from March 15th to June 30th, is 10.51 in.

Transpiration.—The author states that the mean daily rate of transpiration on Chewaucan Marsh was 198% of the green weight of sugar grass on June 27th, which was the 105th day of its growth, and that the plant was fully grown. Is there any reason to assume that the plant stops transpiring water at maturity? It would be just as logical to say that man did not require food and water after he was fully grown. Certainly, transpiration continues until the crop is harvested, which, the writer infers from Figs. 15 to 20, and elsewhere in the paper, is not done until some time in August.

The determination of the 198% is based on 15-min. weighings during 10 hours of a single day in the season. The remainder of the 24 was determined by guess. The weight of green matter was "considered" to increase uniformly during its period of growth. The author states that:

> "Although the experiments seemed to show that the rate of loss immediately after cutting was from 40 to 50% greater than after a period of 15 min. * * *"

this was ignored. Yet the whole process is founded on the theory that the rate of transpiration immediately after cutting is a measure of the rate during growth.

Furthermore, the diagram, Fig. 10, indicates that the so-called cured hay was still losing moisture at the rate of 10% per day. Surely his average crop of 2½ tons per acre is not based on a 3-days' curing. Hay is not usually measured for crop statistics until after 30 days or more in the stack. If measured in this way, it may lose as much as 80% of its green weight.

Cured hay was found to contain 40% of moisture. Hence, it is found that in Chewaucan Marsh the transpiration ratio for green hay was 30% of that of cured hay, and that of cured hay was 60% of that of dry matter.

Now examine the transpiration figures for the Harney Valley, given in each of the diagrams, Figs. 15 to 20, inclusive. The results are given in Table 16. Mr. Stevens.

TABLE 16.

Crop.	Transpiration Ratio to:		
	Green weight.	Cured hay.	Dry matter.
Fig. 15—Sugar and mixed grasses	123	409	682
" 16—Mixed grasses	73	242	403
" 17—Sugar grass	164	597	912
" 18—Sugar and mixed grasses	122	402	669
" 19—Mixed grasses	71	238	398
" 20—Sugar grass	104	346	577

The figures in Table 16 show that, with the exception of Fig. 17, the ratio for green weight is 30% of that of cured hay, and that of cured hay is 60% of that of dry matter. In other words, the cuttings from the several pans were neither cured nor dried. Instead, the green cuttings were weighed, then a factor, determined on Chewaucan Marsh, 100 miles away, on a bright day in June, was used to find the transpiration ratio for different varieties of grasses in the Harney Valley, and the results were given to the Engineering Profession as scientific data.

As the author does not give his figures for plant use, in inches, the writer had to do some guessing. He "guessed" that the author's figures for the transpiration ratio of dry matter more nearly represented that of cured hay, and that the plants transpired water until harvested. The average of the author's ratios for dry matter for the Harney Valley was 600:1, and the writer "guessed" that 650:1 would be about right for Chewaucan Marsh. In this manner the 8 in. and 14½ in. for Harney and Chewaucan, respectively, given at the beginning of this discussion were found.

Soil Absorption.—The writer wishes to protest against the misconstruction placed on the results obtained by Charles H. Lee, Assoc. M. Am. Soc. C. E., as shown by the following quotation from the author:

"From the writer's previous work in soils of a similar nature, it was found that, with ground-water at a depth of about 8 ft., soil evaporation would be negligible, and, for any depth to ground-water of less than 8 ft., the quantity of evaporation would be proportional to the depth. A similar conclusion was reached by Mr. Lee in his work previously cited."

Mr. Lee's experiments were made in order to determine the soil evaporation from the floor of the Owens River Valley, and that evaporation includes transpiration of the native salt grass. His results give the soil losses from the sandy clay loam soils, but Mr. Hammatt has applied them to peat.

Mr. Stevens.

Now, there are peats and peats. The 25th Annual Report of the Vermont Agricultural Experiment Station, 1911-12, contained the following data:

Turfy peat	weighing	7-16	lb.	per.	cu.	ft.,	air-dried.	
Fibrous	"	"	15-42	"	"	"	"	"
Earthy	"	"	25-56	"	"	"	"	"
Pitchy	"	"	38-65	"	"	"	"	"

The author gives the saturation content of peat soil as 274% by weight, of which 253% is free moisture. He also states that the Chewaucan peat soil had 58% voids. These figures give a weight of 14.3 lb. per cu. ft. of air-dried soil, which indicates that the Chewaucan soil would probably fall under the "Turfy peat" classification. The writer's general knowledge leads him to believe that this peat is of a coarse, rooty structure, with a large capacity for water, but with low capillary properties compared with the clay loams. If this is true, the height to which water will rise by capillarity is very limited indeed.

The limit of the rise of water by capillarity in certain soils is given in Table 17.*

TABLE 17.

Soil.	Limit of rise.	Time required.
Sandy soil, Stanislaus County, Cal.	17 in.	6 days
Gila River alluvial soil.	47 "	125 "
Silt soil, Ventura, Cal.	50 "	195 "
Adobe soil, Alameda County, Cal.	46 "	195 "

The writer has seen no similar data for peat soil, but it is very doubtful if water will rise by capillarity more than 2 or 3 ft. When the ground-water falls below the limit of capillary rise, soil evaporation ceases.

On the assumption that the limit of capillary rise in peat soil is 30 in., instead of 96 in., it appears that the author's quantity of sub-soil evaporation, as indicated in Table 12, is 50% too high.

The writer is unable to understand Fig. 23. Apparently, 13½ in. of water were put in the tube on September 11th. Thereafter, at stated intervals, the recession of water in the tube, or the depth to ground-water, was measured. Apparently, at the close of the experiment, the soil had absorbed 10.5 in. of water, which resulted in lowering the ground-water 18 in. In other words, the porosity of the unwatered soil in the tube was 58 per cent. Were the dotted lines merely drawn to confirm the previous determination? Does the diagram really show anything more?

* As given by Professor Hilgard, in "Soils," The Macmillan Company.

Mr. Stevens.

Quantity of Water Actually Consumed on Chewaucan Marsh.—The quantity of water actually consumed is the crux of the whole matter. If the author's processes are logical, his deductions as to the water consumption should conform to the actual consumption, as indicated by the difference in flow at the upper and lower ends of the marsh. The writer has stated (page 253) that:

"This consumption should be comparable with the sum of the first three component parts of the water duty previously given, and, in the writer's belief, will be considerably less than the 37.2 in. which the author's analysis seems to indicate as the required quantity."

Since that was written, *Water Supply Paper No. 410* has come to hand, and it gives the results of stream measurements in the Chewaucan River Valley for the seasons of 1914 and 1915. The writer has also secured the unpublished results for 1916 and 1917, which may be had by any one on application to the Director of the U. S. Geological Survey, Washington, D. C.

The map, Fig. 1, should be consulted in order to understand the situation properly.

Three gauging stations were maintained on Chewaucan River:

1.—The Paisley Station, which is above all important irrigation diversions, 2½ miles up stream from the Town of Paisley;
2.—The Narrows Station, at the constricted portion of the marsh; and
3.—The Hotchkiss Ford Station, just below the marsh.

The following stations were also maintained at the diversion points of ditches watering the alluvial fan at the head of the marsh:

Conn Ditch, irrigating 600 acres;
Smalls Creek, irrigating 2 417 acres, including 1 209 acres under the Bageley Ditch;
Jones-Innis-ZX Ditch, irrigating 2 118 acres.

There are 14 300 acres of marsh and about 6 200 acres of upland watered between the Paisley and the Narrows Stations, of which 5 235 acres are accounted for by the ditches just mentioned. There are also 7 800 acres of marsh between the Narrows and Hotchkiss Ford Stations.

In order to determine the quantity of water consumed on the marsh, it is necessary to deduct a portion which is diverted to irrigate the uplands. A considerable portion of the diverted water finds its way by percolation and waste to the marsh.

In Table 18 only 40% of the water diverted has been considered as consumed; the remaining 60% being returned to the marsh. The diversions for the 965 acres unaccounted for have been neglected. This gives the consumption on the marsh the benefit of all doubt.

Mr. Stevens.

The period of consumption, given by the author, March 16th to June 30th, inclusive, has been used.

TABLE 18.—Water Consumption on Chewaucan Marsh.

	March 16th to June 30th, Inclusive.				
	1914	1915	1916	1917	Average.
Flow at Paisley, in acre-feet	94 600	39 510	77 200	93 640	76 240
Deduct for consumption on 5 235 acres of upland —40% of measured diversions	8 000*	7 900	8 200	7 600	
Net flow to marsh	86 600	31 610	69 000	86 040	
Flow at Narrows, in acre-feet	70 800	11 260	43 200	64 180	
Consumed on 14 200 acres of marsh	15 800	20 350	25 800	21 860	
Depth, in feet	1.10	1.42	1.80	1.52	1.46
Flow at Hotchkiss Ford, in acre-feet	59 250	2 160	31 360	51 180	
Consumed on 7 800 acres of marsh	11 550	9 100	11 840	13 000	
Depth, in feet	1.48	1 16	1.52	1.67	1.46
Consumed on 22 100 acres of marsh	27 350	29 450	37 640	34 860	
Depth, in feet	1.23	1.33	1.70	1.57	1.46

* Records incomplete—estimated.

The period of record includes two years of abundant supply, 1914 and 1917, and also one low year, 1915. The consumption during 1916, when the author's investigations were made, is seen to be the highest of record, although the water supply was about the average. The supposition is, therefore, that the crop conditions during the period of his investigation were somewhat better than the average. If this is not true, the conclusion is inevitable that the hay lands received too much water. At all events, it is obvious that the crops received all the water that was necessary for their growth.

Before making a comparison of the author's computed requirements and the water actually consumed, it is only just to reduce the quantity for plant consumption previously given by the writer, to that represented by the author's deduced transpiration ratio of 347 lb. of water per pound of cured hay. Making this correction, the requirements as computed by the author are:

	Chewaucan Marsh.
1. Soil moisture	12.17 in.
2. Soil evaporation	10.51 "
3. Plant use	7.64 "
4. Soil losses	23.00 "
Total depth	53.32 in. = 4.44 ft.

Now, the first three of these items should be comparable with the measured consumption, if the author's hypotheses and deduc-

tions are rational. That they are not, is evident from the following: Mr. Stevens.

	Author's deduced requirement.	Actual measured consumption.
Water consumption on 22 100 acres of marsh for soil moisture, soil evaporation, and plant use, in feet.....	2.53	1.46
Error	73%	....

The author's Item 4, "Soil Losses," is treated under the caption, "Percolation and Run-off." He states that there is no percolation, but that the marsh hay will not thrive unless the ground is completely covered with running water during the period of growth, and further fixed the necessary quantity as 23 in. of depth.

The actual run-off from the 22 100 acres of marsh is that measured at Hotchkiss Ford, as given in Table 19.

TABLE 19.—Run-off at Hotchkiss Ford, from March 16th to June 30th, Inclusive.

	Acre-feet.	Depth, in feet.
1914	59 250	2.67
1915.....	2 160	0.13
1916	31 360	1.42
1917	51 180	2.31
Average.....		1.62

It is inconceivable that all the run-off is necessary. If this were true, there could scarcely have been any crop at all in 1915, as the run-off was only 1 in. Furthermore, during 1916, while the investigations were under way, it is seen that the author's requirement is 35% greater than the total supply available for this purpose. Little more need be said. The actual figures refute the author's processes and conclusions.

S. T. Harding,* Assoc. M. Am. Soc. C. E.† (by letter).—Mr. Hammatt has presented the results of considerable detailed work on some of the factors affecting the use of water for meadow lands. The irrigation of native grass meadows and pastures by methods involving more or less continuous application of water is found in many localities throughout the West where climatic conditions make the production of crops of higher value uncertain. The nature of the field methods used makes it difficult to segregate the use of water on small areas for experimental purposes without introducing artificial conditions, and it is noticeable that State and Federal agencies have done very little experimental work on such crops. Mr. Harding.

* Berkeley, Cal.

† Now M. Am. Soc. C. E.

Mr. Harding.

To meet these conditions, Mr. Hammatt adopts a method which gives him control of some of the elements observed. In any experimental work involving the duty of water, two conditions must be secured in order to get results which can be applied to field conditions. The first of these is to be certain that the conditions of the experiment are similar to those in the fields; the second is that all essential elements are subject to control and measurement. The use of small volumes of soil in tanks satisfies the second of these considerations, but it may be open to question in regard to the first, although Mr. Hammatt has reduced the differences from field conditions to a minimum.

In many respects the practice described by the author resembles that used in rice irrigation in California. The water is held on the land during a large part of the growing season and is subject to similarly varying evaporation and transpiration losses during the different stages of growth of the rice. Owing to the method of preparing rice lands in checks, the waste or run-off is much less than that estimated by the author for sugar grass. As far as the writer can ascertain, no results of similar observations on rice under field conditions have been published, although the lack of percolation on many rice lands offers an opportunity for observation of evaporation and transpiration during the different stages of growth.

The duty of water depends, not only on the quantity required by plants for their growth, but also on the quantity which it is necessary to use under reasonable standards of practice in order that that required for direct use can be made available to the plants at the time needed. This latter requirement may result in the use of quantities considerably in excess of the direct crop requirements where the conditions are such as to cause large surface waste or deep percolation losses. The only way in which the efficiency of application can be determined is under field conditions. It is noticeable that the detailed work by the author only covered 60% of the total recommended use, an estimated allowance of 40% being included for surface waste.

Mr. Hammatt adds 33% to his estimated evaporation from a deep tank in order to ascertain the evaporation from a thin sheet of water such as occurs on the marshes. The results secured by R. B. Sleight, Jun. Am. Soc. C. E., at Denver, Colo.,* indicated an evaporation 11% greater in June from a tank 3 in. deep than from one 5.75 ft. deep, and of 12% less in October. Mr. Sleight's tanks were more comparable than those used by Mr. Lee, on which Mr. Hammatt's estimate was based. Mr. Sleight found that the placing of a shallow tank above ground gave a higher evaporation, by more than 20%, than a similar tank set in the ground. Mr. Lee's shallow tank was set largely above the ground.

The author's statement, that "the ratio of the quantity of water transpired to the quantity of plant matter produced * * * is within

* *Journal of Agricultural Research*, July 30th, 1917.

such close limits as to be to a certain extent measurable", and the results given relative to transpiration for various standard crops, are, in the writer's opinion, more likely to be misleading than helpful, if an attempt is made to apply them in general irrigation practice. When yields may be varied 100% by various fertilizer treatments for soils receiving equivalent moisture, as under humid conditions, it is evident that the "transpiration ratio", so called, gives little guidance as to the proper duty of water. Mr. Harding.

Mr. Hammatt states that his previous observations had corroborated Mr. Lee's conclusion that, with ground-water at a depth of about 8 ft., soil evaporation would be negligible, and, for any depth to ground-water of less than 8 ft., the quantity of evaporation would be proportional to the depth.

It is the writer's opinion that the results presented by Mr. Lee* do not warrant this as a general conclusion, and that the results of other published experiments show that the depth from which water will be drawn to the surface depends very materially on the texture of the soil. Unfortunately, descriptions of the soil, as given in the reports of some of such work, are too indefinite to permit of making full use of the results.

In order to compare observations of soil evaporation made in different localities, some means of correlating climatic factors is required. The evaporation from a water surface is probably the best single unit for this purpose. In Fig. 24 a number of such observations have been assembled, the evaporation from the soil being expressed as a percentage of that from a water surface at the same locality.

Five of the results given are from Mr. Sleight's work at Denver, reference to which has been given. Mr. Sleight defines the texture of the material by the size of the opening of a screen, in inches, such that 60% of the material passes and 40% is retained. The tanks used were 2 ft. in diameter, and the results have been compared with the evaporation from a water surface in a tank of similar size. The coefficients which should be used in reducing the evaporation from soils in tanks to that from large areas are not known; some limited experiments by Mr. Sleight indicate a more marked variation from soils than from water. The four coarser materials are river-bed sands; the fine sandy loam is from the soil at the laboratory.

The results given in *Bulletin 52* of the Wyoming Experiment Station were secured in 6-in. tanks containing soil cores taken in the field. The texture of the soil is not given.

In his work, Mr. Lee states that No. 2 tank was filled by replacing the soil as found with the stratification as existing in the adjacent soil.

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 148.

Mr. Harding.

Tank No. 3 had a layer of black loam soil under the sod, the remainder of the soil being mixed. It is stated that Tanks Nos. 4 to 7 contained a mixture of clay, soil, and fine sand, a rather general description. A curve based on Tanks Nos. 4 to 7 only, omitting Tanks Nos. 2 and 3, which do not plot consistently with the others, would, if extended, indicate a zero evaporation for this mixed soil when the water-table

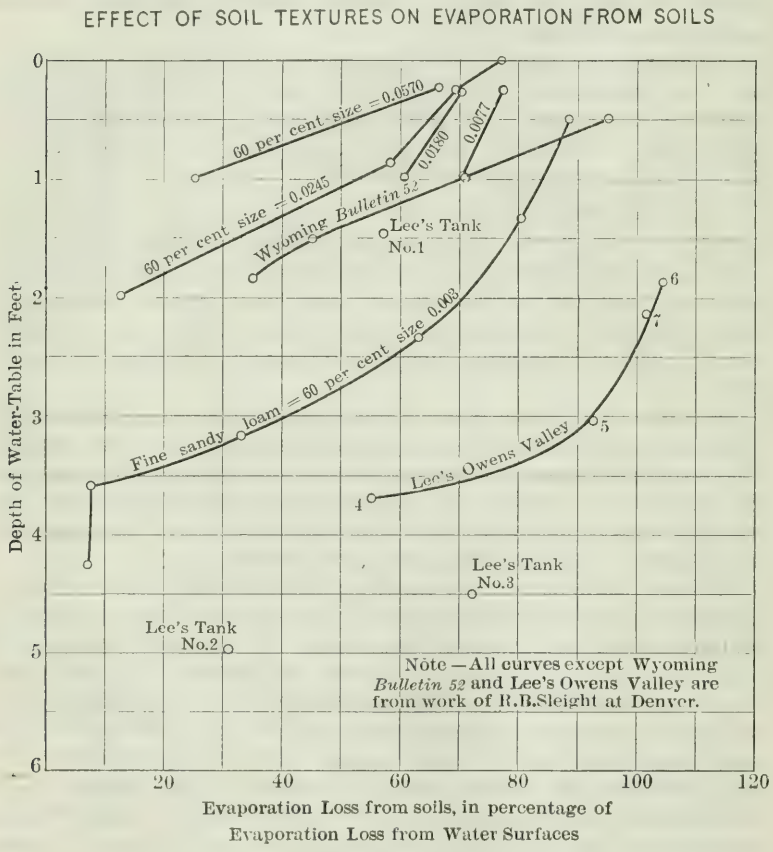

FIG. 24.

is somewhat more than 4 ft. below the surface. Mr. Lee's soil tanks were 7½ ft. in diameter, his deep-water tank was 3½ ft. in diameter, and would have a coefficient of 0.83, according to Mr. Sleight's experiments. This coefficient was applied to the evaporation from the deep-water tank, as the larger size and growth of sod would probably give a small correction for the soil tanks.

Mr. Harding

Mr. Lee's Tank No. 1 was without sod, but had on the surface 2½ ft. of clean river sand of mixed sizes. The results for June to September, 1910, are given in *Water Supply Paper No. 294*, and are plotted on Fig. 24.

From the curves on Fig. 24 several conclusions may be drawn. The most evident one may be the need of more thorough investigations of this subject, giving more consideration to the texture and arrangement of the material. It also appears, however, that the depth from which moisture will be drawn to the surface, and the extent of evaporation from different depths, varies very materially with the texture of the soil. For coarser materials, the depth from which evaporation will occur may be less than 2 ft.; data are lacking on soils of heavy texture. The fine sandy loam at Denver indicates little loss for depths of more than 5 ft. on such medium to light soils.

The results of Mr. Lee's work are the only ones given for a soil supporting vegetation. The increased proportion of loss from his Tanks Nos. 4 to 7 may be due to the heavier texture of the soil or to the additional effect of the salt grass roots in drawing water for transpiration, the description of the soil is too indefinite to enable one to compare it with the other experiments.

The results in general indicate that in coarse material, with the water-table 3 in. below the surface, the evaporation will be less than from an adjacent water surface; with medium or heavy soils it may equal or exceed such loss from a water surface.

The evaporation from soils is often an important item in matters involving the loss from marshes, the salvage in water which may be effected by drainage, the water requirement of such crops as those included in Mr. Hammatt's work, and in the saving that may be secured in cultivation with soil mulches. The field of investigation is one requiring much careful experimental work and analysis of results, the data available are limited, and there is much opportunity for additional work.

Mr. Hammatt.

W. C. Hammatt,* M. Am. Soc. C. E. (by letter).—The writer is pleased with the discussion brought forth by his paper, but somewhat disappointed that so little of it had to do with the methods used, and that the main part merely bore on the results obtained on the particular work cited. The intention of the paper was to introduce for discussion and criticism, with a view to improvement in method, the principle of analytical experiment as applied to the determination of the duty of water in irrigation. The two campaigns carried out by the writer in Lake County and Harney County, Oregon, were merely given as examples, with apologies for defects in the methods used, in some cases due to lack of time and necessity for hurried results.

* San Francisco, Cal.

Mr. Hammatt.

The discussion by Mr. Versluys is interesting in showing results obtained in rice culture in Java, rice being the one cultivated crop comparable in any way with the wild grasses on which the writer's experiments were made. The writer, however, is somewhat surprised at the results, as the average yield—1 000 lb.—is only about 40% of the average yield for California. Possibly this is due to the extreme rainfall conditions in Java, the average annual rainfall varying from 72 to 175 in., distributed throughout the entire year. This would have a tendency to increase the quantity of straw at the expense of the grain, allowing an insufficient dry period for maturing the grain. It is probable that if the rainfall were properly distributed no irrigation whatever would be needed, as the total annual rainfall is in excess of the requirement for rice.

The average quantity of water used in the Sacramento Valley in California for rice culture is about 99 in., the extremes ranging from 53 to 179 in. Of this only about 1½ in. was precipitation during the irrigation season. The irrigation season, by the way, is from the middle of April to the last of October, ranging from 147 to 189 days.

The work of investigation in Texas, mentioned by Mr. Rockwell, will doubtless be of great interest, and the writer hopes that full descriptions of the methods used and results obtained will be published for the benefit of the Profession. The writer judges the sampling at various depths and intervals to be for the determination of the distribution of moisture in the soil both during the penetration of the irrigation water and its consumption by evaporation and transpiration.

Mr. Harding's criticism of the addition of 33% to deep-water evaporation rates on account of shallow water is well founded. Mr. Lee's experiments do not justify this as a fixed percentage, and the quantities given by Mr. Sleight are probably more correct for the conditions obtaining on Chewaucan Marsh during the irrigation season, if the increment is considered as due only to the water depth. There are other conditions obtaining on the marsh—which are not encountered in the pan experiments—which tend to increase the evaporation rate, so that, in the aggregate, the 33% increment is probably justified. The condition having the greatest influence is that of the dense plant growth, causing the water to be drawn up the outside of the stalks and the evaporation to be accelerated.

The exception taken by Mr. Harding to the writer's statement that "the ratio of the quantity of water transpired to the quantity of plant matter produced, * * * is within such close limits as to be to a certain extent measurable", is justified on the basis that he makes in his explanatory statement. The writer, however, only intended to apply it to cases where other conditions were equal. In the case of forage crops, where the plant food is taken largely from the air, it holds very closely

through a wide range of soil conditions. A seed crop, however, will have a great variation in transpiration ratio, depending on the soil fertility. Mr. Hammatt

As Mr. Stevens makes specific objections to several of the results obtained by the writer, his various points will be answered in the order in which they occur in his discussion.

The writer had no knowledge of the 6-year record of evaporation at Keno, Ore., at the time of the investigation on Chewaucan Marsh, or, for that matter, at the time his paper was written. Inquiries at the Oregon Agricultural College, the U. S. Geological Survey, and the U. S. Weather Bureau succeeded only in unearthing the Ady record. Had he known of the Keno record, he would have made use of it in preference to the Owens Valley record.

As for the discrepancy between his tabulation of the Ady record and that published by Moritz, the writer did not, as stated by Mr. Stevens, quote Moritz as his authority, except for those months specifically mentioned. The writer's figures were obtained from photostat copies of the original reports, furnished by the Weather Bureau. As for the difference between the results of the Ady record and that at Keno, the writer cannot agree with Mr. Stevens that it was due to difference in water temperature.

The writer cannot believe that the water temperature has in itself any great influence on the rate of evaporation, within the limits of temperatures usually occurring. According to well-known physical laws, a definite volume of air, at a particular temperature and density, will absorb a definite volume of water vapor, this being dependent on the temperature and density. As saturation is approached, the rate at which the vapor will be absorbed will of course diminish, hence the effect of humidity. If the water is in thin sheets, spray, or capillary form, evaporation will be accelerated. To liberate 1 lb. of water vapor, about 966 B.t.u. must be absorbed by the water. In natural conditions, this heat is absorbed from the atmosphere, which is thus the controlling factor.

Mr. Sleight, in his experiment, placed an electric globe in some of his pans to control their temperature. He thus made the experiment entirely dissimilar to the natural condition of evaporation, supplying the heat necessary to evaporate the water by an artificial heating plant. The fact that the evaporation was greater in the heated than in the unheated pan merely proved that a greater quantity of heat was being supplied to it, which fact was corroborated by the rise in temperature of the water. Under natural conditions, the temperature of the water surface is controlled by the evaporation, rather than the converse. Mr. Sleight did not state in his report the size of the electric globes used, nor the time they were kept lighted. A 40-watt lamp will produce about 3 260 B.t.u. per 24-hour day, sufficient for the evaporation of 3.38 lb. of water, which, in the tanks used in the experiment, would be

Mr. Hammatt.

equivalent to 2.06 in. in depth. A considerable portion of this heat, however, is dissipated through the walls and bottom of the tank.

Also, in the experiments to determine the effect of flowing water on evaporation, the conditions of the experiment were totally unlike any natural conditions. The water was circulated with a centrifugal pump, and, from an inspection of the single record published, the work done by this pump was sufficient to raise the temperature of the water 10 degrees. Although a correction was made for the difference of water temperature by comparison with pans heated by electric globes, it is extremely doubtful whether the relative absorption and evaporation from the heat generated by the bulbs would be the same as from that produced by the working machinery. It is the writer's opinion that, under similar conditions, there is likely to be greater evaporation from flowing than from still water, due to surface agitation and to friction with the atmosphere, but this will be an indeterminate variable, and due rather to the relative movement of the wind and water than to the absolute motion of either.

Mr. Stevens states that "the correct figures for evaporation at Ady are already 40% too high, yet the author increases them by 7½% for differences in elevation." If the figures of the U. S. Geological Survey at Keno are accepted, as against those of the Weather Bureau at Ady, which is questionable, the difference is 29% instead of 41 per cent. The effect of elevation, or atmospheric density, however, seems to be open to some question.

Soon after the publication of this paper, the writer had his attention called to the statement that "evaporation at different elevations is in inverse proportion to the barometric pressure." It was immediately apparent that the principle was stated improperly, and he wrote a short discussion correcting the error and making additional explanations of the matter. It was decided, however, to withhold this discussion until the writer submitted his closure; the substance thereof is given below. The statement should have read as follows:

"The writer has always believed that, other conditions being equal, the evaporation will vary with a function of the difference in barometric pressure at the different points."

The writer based his opinion and belief on the fact that the boiling point of water lowers as the barometric pressure decreases, requiring a smaller application of heat to vaporize the water at higher elevations. It is true that the data published in the paper by Messrs. Duryea and Haehl are insufficient to establish this beyond question, but a platting of the data in Table 17 of that paper shows that the curve of evaporation relation coincides with that of barometric relation between the limiting elevations of 2 500 and 5 000 ft. As Dr. Fortier's experiments ranged from Elevation 4 500 to 14 500 ft., they are nearly all above those quoted by Messrs. Duryea and Haehl, so that it is possible that the

relation may change at the higher elevations. The writer's confidence in the physical law quoted is such, however, that he would rather be inclined to study with care all the conditions attending the Fortier experiments in order to ascertain if there was not a possibility of other influences being strong enough to counteract that of elevation. Mr. Hammatt.

The specific correction for the influence of elevation on the rate of evaporation by comparison with the evaporation records in Owens Valley was made from a study of the figures in Table 20 showing the barometric differences, considering the barometric pressure at sea level as 30 in. of mercury.

TABLE 20.

Elevation, in feet.	Barometric height, in inches.	Drop in barometer from sea level.	Percentage relation to barometric height at Elevation 3 775.
2 000	27.79	2.21	— 44.8
3 000	26.74	3.25	— 18.5
3 775	25.96	4.04	.....
4 000	25.74	4.26	+ 6.4
4 090	25.65	4.35	+ 8.6
4 410	25.34	4.66	+ 16.3
5 000	24.77	5.23	+ 30.6

The writer makes no claim that the various coefficients used in correlating the evaporation records at Independence to conditions existent on the Chewaucan Marsh are absolute. He maintains, however, that their principle is established by physical laws, and that their application is a matter of engineering judgment in connection with careful observation of field conditions. This is particularly true with the deduction for plant cover, characterized by Mr. Stevens as "mere guesswork." The physical law is that saturated air will take no additional water vapor, and must be replaced by dry air if evaporation continues. The fact, determined by the writer by inspection at various periods of growth, was that, as the grass grew higher and spread, the movement of air was more and more restricted. Although the straight-line variation may not be justified by experiment, it is by logic, until experiment shall have given a better one.

In this matter it is interesting to note the actual results of the soil pan evaporation tests in Harney County, as shown in Figs. 15 to 20, inclusive. These are tabulated in Table 21, from which it will be noted that, although the season averaged shorter than that assumed for the Chewaucan calculation, the total evaporation was greater. This may be explained by the fact that the season falls about a month later in the Harney County tests than in the Chewaucan calculation, a part of the hotter months—July and August—being included in place of March and April, used in the Chewaucan figures.

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TABLE 21.

Location.	Year.	Length of Season.			Evaporation loss, in inches.
		Begins.	Ends.	Days.	
Wickes Field	1915	April 8	July 13	96	12.10
Frye Field	1915	May 18	July 15	58	8.90
Verdo Field	1915	May 5	July 12	68	7.50
Wickes Field	1916	March 30	August 4	127	15.78
Frye Field	1916	April 13	August 4	114	15.62
Verdo Field	1916	March 28	August 3	128	17.33
Mean				98.5	12.87

In Fig. 25 are platted the heights of the growth in the various pans in the Harney County tests during the season of 1916. The uniformity of growth is apparent from this diagram, justifying the straight-line reduction of evaporation for plant cover and the uniformity of transpiration increase.

Mr. Stevens states that plants continue to use water after maturity, and compares the use of water by plants with the consumption of food by man, with a fine disregard for the laws of mechanics. The food consumed by man is transformed into mechanical energy, which is dissipated in work or motion. Plants perform no such work, and their food goes entirely into growth. When a plant reaches maturity, it ceases to take in water through its roots, and loses in drying a large portion of that already taken, making hay while still standing. There is even a claim of a slight backflow into the earth, causing the loss of some of the food value, for which reason it is considered advisable by stockmen to cut for hay slightly before maturity, though the writer is somewhat skeptical as to this, and believes that the greater food value of hay cured after cutting over that cured standing is probably due to other causes.

Mr. Stevens is evidently not familiar with the practice of curing hay. It is not usual to leave the hay on the ground more than 48 hours, and where possible it is stacked within 72 hours after cutting. Stockmen consider that the hay loses in food value if left longer without stacking. Moreover, after putting in the stack, it not only ceases to lose weight, but gains in weight by absorption from the atmosphere. Mr. Stevens' inference that the ratios of green weight, cured weight, etc., used in the Harney Valley experiments were taken from the test made at Chewaucan Marsh is also in error, the cutting from each of the pans having been cured for 48 hours and weighed at intervals during the period.

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The writer, after considerable study, is unable to comprehend Mr. Stevens' "guessing" paragraph, so assumes that it was put in to add humor to his discussion.

Regarding the matter of soil absorption, Mr. Stevens' information is correct as to the nature of the peat, and also in regard to its low capillary limit. The layer of actual turfy peat, however, is quite thin, being from 8 to 12 in. in depth, followed by a mixture of peat in the white volcanic soil, described in the paper as underlying the marsh, and gradually changing to the unmixed white soil. Although the turfy peat has a low capillary limit, it shrinks and cracks in drying, allowing free access of air to the underlying soil, which has a much higher capillary limit. Test holes bored in September, 1916, two months after the discontinuance of irrigation, showed that the soil was practically dry for a depth of 2 ft. About thirty holes were bored at this time,

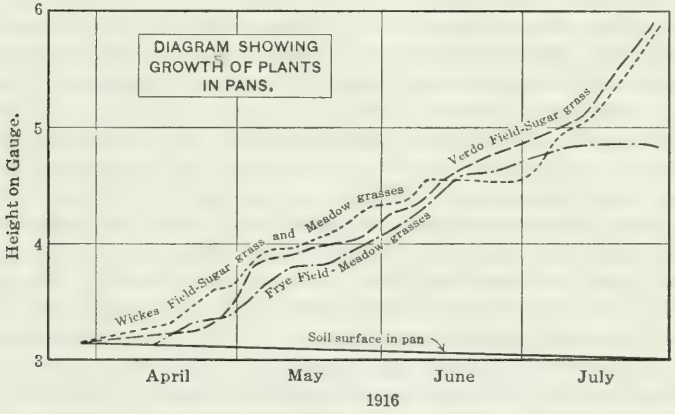

FIG. 25.

representative of the entire area of the two marshes. As the subsoil has been found to be practically impervious, the contained water at the close of irrigation must have been lost by capillary rise and surface evaporation.

In the later test of the absorption capacity of the soil, Mr. Stevens correctly interpreted Fig. 23 as showing the putting of 13.54 in. of water into the tube on September 10th, 10.15 in. being introduced at 4.30 P. M. and 3.39 in. at 5.05 P. M. The top of the tube was covered to prevent evaporation, and measurements of the lowering of the water surface were made at intervals thereafter. The curve above the line, showing the depth of water absorbed, was platted from the observations up to September 28th, the broken line being produced at the same rate, which had been uniform for the previous two weeks. The curve below the line, showing the depth of penetration of the absorbed water,

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was platted for the first 6 in. from tests taken previously, the final measurement being taken on September 28th, when the tube was removed and the core examined. The line between these points, as well as its dotted extension, was made from the percentage relation found from the measurements taken.

We now come to the interesting part of the discussion, the attempted application of run-off data at the three main gauging stations on the Chewaucan River to determine the actual consumption on the marsh. The arguments used remind one so strongly of those appearing in *Northwest News,* a promotion sheet published by the real estate firm promoting the Paisley Project, and the data presented have such a similarity to those presented by the engineer for that contestant in the litigation, that the reason for the rather acrimonious nature of Mr. Stevens' discussion may be inferred. In fairness, the records of the case should have been quoted, showing the presentation of data to the effect that, although the measurements at Hotchkiss Ford included all water running off the marsh, those at the Government gauging station did not include all that running on the marsh, that the water passing the Hotchkiss Ford gauge did not all pass the Narrows, and that passing the Narrows did not all pass the Government gauge. Testimony was presented showing a very considerable water-shed with a high run-off percentage tributary to the marsh, the run-off therefrom being intercepted by a drainage canal along the edge of the Upper Marsh and delivered at the Narrows, and along the edge of the Lower Marsh, to Hotchkiss Ford. A creek of considerable size, known as Moss, or Avery, Creek, empties on the Upper Marsh and is not included in any measurements given of inflow, while another, still larger, known as Willow Creek, is used for irrigation near the Lower Marsh, and was, by the breaking of dams, diverted into the Lower Marsh without appearing in any measurement except that of Hotchkiss Ford. The effect of these is very apparent by comparison of the daily hydrographs at the three stations, the run-off at Hotchkiss Ford often exceeding that at the Narrows during rainfall periods, and that at the Narrows increasing by a much greater percentage than that at the Government Station.

Since this paper was written, the Court has handed down a decision based on the evidence produced at the trial, allowing a duty of 3.9 ft., or 46.8 in., somewhat smaller than that claimed, but about double that advocated by Mr. Stevens. This decision has been upheld by the Supreme Court, an appeal having been denied. The difference between the duty claimed, 60.1 in. and that allowed by the Court, 46.8 in., or 13.3 in., was doubtless due to the Court's opinion that the full run-off claim of 23 in. was not indisputably established.

The writer acknowledges the impossibility of establishing the exact quantity necessary to insure the proper movement of water over the

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marsh, due to the irregular flow of the river. The total run-off measured at Hotchkiss Ford, shown in Table 19, certainly did not represent the necessary run-off, as a material part of it was run-off from the adjoining water-shed during storms, and did not go on the marsh at all. The actual necessity for the movement of the water is well established, however, an analysis of the plant growth on the marshes having been made by Professor P. B. Kennedy, of the University of California, and it being shown by survey that whenever the flow of the water was impeded by knolls or ridges, the valuable grasses were replaced by tules and cat-tails, and wherever the flooding was intermittent, as on the tops of knolls, covered during the periods of greatest flow but uncovered during periods of lesser flow, the predominant growth was blue-joint and fox-tail. As for Mr. Stevens' statement that if the claim of the necessity were true "there could scarcely have been any crop at all in 1915," he is quite right, there having been no crop at all on those parts of the marsh where the water ran on and was consumed without run-off, the only full crop being obtained from the small acreage at the head of each marsh, where the water was first turned on the land and kept running continuously over it. This point was also well established by testimony, and appears in the Court records.

A study of the hydrographs of the various years of record shows in general an excess of water during the early summer, with a falling below the requirement from the middle of May onward. By turning the whole flow over the land and flooding it to a depth greater than was actually needed, the water was held and the uncovering of the marsh delayed beyond the time when it would otherwise take place. It is this which causes the run-off percentage to be so high in some years. For instance, in 1914 there was a very high March and April flood, causing a high run-off for that year. In 1916, the flow of the water was ideal, coming at the exact time and in the quantities needed, causing a low run-off during the parts of the season when in general the run-off is high.

The writer asks the indulgence of his readers for having gone into so much detail in his closure on a paper which was intended—and so stated in the opening paragraphs—to be merely descriptive of methods used, for the purpose of drawing discussion and criticism of these methods, and tending toward improvement in future studies. It was not his intention to make any point of the results obtained, these being of merely local interest. This point is entirely missed or ignored by Mr. Stevens in his discussion, in which he criticizes the writer's failure to touch on the matter of efficiency.

The discussion, however, has enlarged the writer's knowledge by offering him additional food for thought, and enables him to draw the following conclusions:

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(1).—That laboratory experiments are apt to lead to erroneous conclusions if the exact field conditions are not reproduced. This point was brought out by Mr. Harding. It was to satisfy this condition that the pan experiments in the Harney Valley tests were made in the fields themselves, instead of all together in one place, reproducing the same conditions as to plant cover and water depth in the pan as in the adjoining field. One pan of each set had a portion of the water removed and replaced from time to time, in order to reproduce as far as possible the condition of changing water due to flow on the land.

(2).—That there is a sparsity of evaporation data, and such records as there are lack sufficient completeness of detail to permit comparison. It would be advantageous if all the evaporation work could be conducted by either the Weather Bureau or the Geological Survey, in order to insure uniformity, the pans being run at the principal stations in combination with the other data, in order to have available for every section a record of evaporation as well as of rainfall, run-off, temperature, etc.

(3).—That subsoil losses can only be determined by field experiment on a large scale, and necessary surface run-off can only be determined by long-time observations of results under conditions as they exist.

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Paper No. 1429

CEMENT JOINTS FOR CAST-IRON WATER MAINS*

By Clark H. Shaw, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Harry Y. Carson, F. M. Randlett, Walter Pearl, H. G. Moulton, H. B. Lynch, Edward R. Bowen, and George W. Pracy.

Synopsis.

This paper presents the method of making successful cement joints in cast-iron water mains.

A brief history of the use of this joint establishes the fact that this method of construction has long since passed the stage of experiment, and has been proved to be an economic factor in laying such mains.

The process of making the joints is described and illustrated, some experiments in jointing are described, cases showing the strength of cement joints under trying conditions are cited, and data relating to cost, etc., are presented.

About 1886 a cast-iron pipe line for water distribution was laid with cement joints at Redlands, Cal., and in 1891 joints of that kind were used at Los Angeles, Cal., but evidently with questionable results, as the method was not adopted. In January, 1907, Mr. Charles Thornburg, then Superintendent of one of the water companies operating in Long Beach, Cal., decided to try cement joints for a 16-in.

* Presented at the meeting of April 18th, 1917.

cast-iron pumping main, and instructed his foreman of construction, Mr. F. M. Shrode, to conduct some experiments.

No definite process was outlined to the foreman, but his experiments and practice in repairing steel riveted water mains under pressure, by using a dry mixture of neat cement in caulking the bands around these pipes, gave him an idea that a moist cement could be caulked into the bell solidly, and would produce the results desired. It was probably Mr. Shrode, who, by this experiment, finally perfected the joint and used it in the construction of the entire line. When this line was completed and put into service, working under a static head of about 190 ft., several places showed some seepage, particularly at the lower end of the line, where the work was started, and it was decided to re-caulk these joints at the first opportunity; it was noticed, however, that the moisture was gradually drying up, and the seepage finally ceased.

Cast-iron construction was then abandoned by this company until 1911. During that year the works came into the possession of the municipality, and the writer was appointed Engineer of the Water Department. After looking into the merits of the cement joint, as used on this 16-in. pumping main, it was adopted as the proper method of construction, and since that time it has been used throughout the entire system.

Long Beach now has 60 miles of cast-iron water mains, ranging from 4 to 24 in. in diameter, laid with joints of this type. All these pipes are under pressures ranging from 40 to 80 lb. per sq. in., and are giving perfect satisfaction.

Method of Making the Joint.

In making the cement joint, the pipe is placed and spaced in the usual manner. A thin backing of the best dry jute is used instead of oakum, as the jute is free from oils and grease (which should be avoided). A Portland cement, conforming to the specifications advocated by the American Society for Testing Materials, is used. The dry cement is placed on a piece of canvas (usually a cement sack ripped open), and moistened just so that when thoroughly mixed by hand (Fig. 1) it will be of such a consistency that when gripped tight it will hold the form of the hand (Fig. 2), and when dropped 12 in. it will crumble (Fig. 3). The canvas containing the cement is placed

under the bell, and the cement is tamped into place by hand with a caulking iron until the bell is about half full (Fig. 7). It is then caulked with heavy blows until the cement is thoroughly packed in the back of the socket. This process is continued until the bell is packed solid out to the face (Fig. 8). A small bead of neat cement in a plastic condition is then put on, using the caulking iron as a trowel (Fig. 9). As soon as the initial set of the cement in the bead has taken place, the joint is covered with earth to protect it from the air and sun. In back-filling, the excavated material is always settled with water, which helps to cure the exposed portion of the joint.

The bead is essential, in the writer's opinion, as the cement packed in the bell is so dry that without protection it would absorb moisture from the water used in settling the trench, and it is believed that, should the joint develop seepage when the pressure is put on in the main, the cement, being dry, would expand and aid materially in keeping the joint tight.

Experiments on cement joints constructed without the bead showed that, 24 hours after completion, they absorbed water readily. In cases where seepage has developed and has subsequently stopped, it is assumed that the dry cement absorbed the moisture from the inside, expanded, and filled the seepage pores.

About 20% of the cement is wasted by falling off the canvas or being thrown out by the caulker. If any dust or earth from the trench falls on the canvas or in the cement, it is immediately taken out, together with enough cement to make sure that the remainder is clean. In mixing the cement with water, care is taken that there shall be no lumps in the material, no matter how small. If any cement is left on the canvas when a joint is completed, it is used on the next joint, provided the work is continuous, otherwise new batches are made. Special blunt caulking tools are used (Fig. 4).

The joint is allowed to stand 48 hours before the pressure is turned on and the main is put into regular service. Cement joints have been used with satisfactory results, however, 12 hours after completion, but this is not considered safe practice. Pressure tests are never made by the writer prior to putting a main into service.

At San Diego, Cal., a pressure test was made by caulking a 6-in. cast-iron tee, one side of the tee being filled with a plug and each of the two ends filled with short lengths of cast-iron pipe with plugs caulked in the ends. As the pieces of pipe caulked in the tee were scrap ends

cut from other pipes, they had no bead on the joint end, and, notwithstanding the fact that the joint was made with smooth pipe, it took a pressure of more than 300 lb. per sq. in. to force the pipe out. The test was made about 48 hours after the joint was made.

In another test, made at Winnipeg, Man., three lengths of 6-in. pipe were laid with four cement joints, on January 13th, 1916. After 6 days, pressure was put on the pipe, in increments of 25 lb., and the joints were found to show no leakage or moisture, up to 125 lb. At 150 lb. one joint showed moisture on the surface of the cement.

On January 24th another test was made, and at one joint moisture appeared at 175 lb. On January 31st this joint showed moisture with 200 lb., and also on March 15th, with a pressure of 255 lb. This joint was the weakest of the four. The pressure was kept on the pipe about one-half hour in each case.

Fig. 5 is a section of a joint made with a cut piece of pipe, and shows the position of the jute and the cement when there is no caulking rim or bead on the end of the pipe (as in the San Diego test). The cement bead, as shown in Fig. 5, was made larger than usual, and covers the entire face of the bell, though normally it covers only about one-half.

The strength and rigidity of the cement joint are shown by the following instances where cast-iron mains have been subjected to severe tests: A new trenching machine was being tested by a sewer contractor in Long Beach, Cal. It operated parallel with and 5 ft. from a 6-in. cast-iron water main, with cement joints, which had been laid in January, 1915. The trench dug by this machine was 3 ft. wide and 18 ft. deep. Some time before noon on February 1st, 1915, the side of the trench next to the water main caved in, leaving about 40 ft. of the pipe hanging and supporting about $2\frac{1}{2}$ ft. of earth on top of it. This condition was not reported until the next morning, and at nine o'clock braces were put in to support the pipe. The main was under a pressure of about 65 lb. per sq. in. at the time. There was not the slightest seepage from any of the joints as a result of this strain.

Again, in June, 1915, 94 ft. of 4-in. cast-iron pipe with cement joints fell into a sewer trench, as the result of a cave-in under conditions similar to those just described. This pipe was under a pressure of about 55 lb. per sq. in. At one end it broke at a service connection;

Fig. 1.

Fig. 2.

Fig. 3.

Fig. 4.

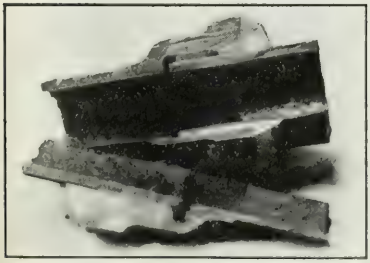

Fig. 5.

Fig. 6.

Fig. 7.

Fig. 8.

Fig. 9.

at the other end it broke near the spigot end of a pipe, about 8 in. from the joint, letting the whole 94-ft. section fall into the trench. In this case every cement joint remained intact. This line had been laid in March, 1913.

Several thousand feet of 8 and 10-in. cast-iron water mains with cement joints have been laid in made ground, the fill being silt from the dredging of harbor channels; also several thousand feet of 8-in. cast-iron pipe have been laid in fine beach sand; all are giving perfect satisfaction. An 8-in. cast-iron main with cement joints was laid in filled ground (the soil being clay) with only 6 in. of covering above the pipe when the line was put in service; the fill was then completed to 18 in. above the top of the pipe, and was rolled with a 14-ton steam road roller, without causing the slightest seepage in any of the joints. The rolling was done preparatory to paving the street in which the pipe was laid. This was a very severe test of the merits of the cement joint. At the time this rolling was done, had any seepage developed, it would have been readily detected, as the whole line was within 3 ft. of the edge of the fill. The fill was completed to a width of 80 ft., 3 months later, leaving the pipe 12 ft. from the center of the street.

In several instances a cast-iron main laid with cement joints has settled 3 in., or probably more, in loose or filled ground, without developing any leakage. In fact, there is only one case that the writer can recall where the cement joint was not satisfactory. This was in a 6-in. cast-iron main on a dock, about 3 ft. from a railroad track. Many joints in this pipe have developed seepage, and some have small pin leaks, but the leakage is not considered serious enough to warrant closing down the line for re-construction. This failure may be due to faulty construction, as the pipe was laid when the making of cement joints was in its infancy. However, the main is situated so that it cannot be entirely covered.

Removing Cement-Jointed Pipes.

Fig. 6 shows a completed joint. The cement joint can be taken apart in a very simple and economical way. The pipe is uncovered about one-half, or a little below the center. At the joint where the original bell-hole was dug, the trench is usually made wider on the sides (but not deeper under the pipe), in order to permit the caulker to work at the joint. The upper half of the joint is cleaned out with

a cape-chisel; then, with tripod and blocks, the free end of the pipe is raised until the lower half of the joint breaks free from the bell. The pipe seldom has to be pulled out of the bell, as it nearly always works itself out as the free end is lowered. If portions of the cement stick to the spigot end of the pipe, or fail to be entirely crushed in the bell, it is a very simple matter to clean out the bell with a cape-chisel, or knock the cement from the spigot with a hammer.

On occasions, after a joint has been cemented tight in the line, it is necessary to cut it out entirely (such as for laying a valve on its side; turning a tee or Y in another direction; adjusting a tee to conform to or meet a grade; avoiding a sewer connection or any other unforeseen obstacle). Table 1 has been compiled from records of the actual time spent in doing such work.

TABLE 1.—Time Required for One Man to Dig Out a Complete Cement Joint, Without Removing the Fitting or Gates from the Line.

Size.	Time.
4 in.	18 min.
6 "	22 "
8 "	26 "
10 "	30 "
12 "	38 "
14 "	48 "
16 "	60 "

TABLE 2.—Data Relative to Cement Joints.

Size of pipe, in inches.	Rings of jute per joint.	Jute per joint, in pounds (approximate).	Number of joints per 94-lb. sack of cement.*	Number of joints per 8-hour day (one caulker).
4	2	0.14	24	50
6	2	0.19	18	42
8	2	0.24	14	34
10	3	0.43	11	28
12	3	0.51	8	24
14	3	0.58	7	20
16	3	0.66	6	17
18	3	0.73	5	14
20	3	0.80	4	11
24	3	0.95	3	7

* Including the 20% of cement wasted or left over.

The writer has never cut out a joint on a main of greater diameter than 16 in. It is fair to assume that to cut out the upper half of a joint, for the purpose of removing a pipe, would take only one-half the time indicated in Table 1.

At Long Beach unit costs have been kept on all construction, covering nearly the entire 60 miles of cast-iron water mains. Table 2 has been carefully compiled from these unit costs, and presents data concerning cement joints.

DISCUSSION

Mr. Carson.

HARRY Y. CARSON,* JUN. AM. SOC. C. E.† (by letter).—It would be a valuable addition to engineering literature if some one were to compile a comprehensive and authoritative paper or thesis on joints for cast-iron pipe. Much has appeared in the technical press, as well as before technical societies, with reference to the leakage of water and gas from underground mains, but, obviously, not enough has yet been done to bring about a general improvement in the prevention of these large percentages of wasted water and gas. This paper is a valuable contribution to such literature as now exists.

The causes for the normal leakage at joints in mains may be classified as follows:

1.—Contraction and expansion;
2.—Unequal settlement;
3.—Vibration and shock.

A fourth cause may be corrosion, but a very able paper, entitled "External Corrosion of Cast-Iron Pipe",‡ by Marshall R. Pugh, M. Am. Soc. C. E., points out that deterioration in cast-iron pipe, as compared to other materials, is practically nil; that the oldest cast-iron pipe, flanged, and in 1-m. lengths, put into service more than 250 years ago, has shown no appreciable loss in wall thickness or strength, and that: "experience has not been sufficiently long to establish just what its life is." The chief consideration for cast-iron pipe, therefore, may be said to center about the type of joint best suited to prevent waste through leakage.

Contraction and expansion exist in all mains, and vary in direct proportion to changes in the temperature of the pipe line. The movement caused thereby, though slight, and scarcely ever exceeding 2 in. per 1 000 ft., is absolutely irresistible, and its effect must be taken care of properly, or disaster is sure to result. No material is of sufficient strength to resist its power if rigidly maintained against it. Contraction and expansion produce by far the greatest proportion of normal leakage in mains. Because of its inelasticity and fragility, cement, as a material for jointing cast-iron pipe, has always been questioned, in spite of the fact that it is very cheap.

The author makes the following statement:

"Long Beach now has 60 miles of cast-iron water mains, * * * laid with joints of this [cement] type. All these pipes are under pressures ranging from 40 to 80 lb. per sq. in., and are giving perfect satisfaction."

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* Birmingham, Ala.

† Now Assoc. M. Am. Soc. C. E.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 806.

Fig. 10.—Suspended Line of Pipes with Metal-to-Metal Joints.

He presents no data, however, by which the satisfactory performance of the line may be measured, as he has not submitted any information as to tests made for leakage or figures as to its quantity. Manifestly, without this information it is difficult to see how the line can be stated to be perfectly satisfactory. Mr. Carson.

The principal advantage of bell and spigot pipe when packed with lead lies in its ability to take care of the always existent contraction and expansion at its joints, without fracture. Cement joints may reduce the number of joints actually leaking, but rigidity is increased to such an extent that there may result an increasing number of fractures at those points. The net leakage with cement joints, however, is probably not materially different from lead.

Flanged jointed pipe, in order to withstand expansion and contraction, must have expansion joints at very frequent intervals.

Regardless of the kind of jointing material used, the unequal settlement of a pipe line may cause normal leakage as well as breakage, with extraordinary leakage. Damage ensues from settlement almost in direct proportion to the relative rigidity of the joint. Flanged pipe is particularly unsuited to conditions where unequal settlement prevails, as is the case in the down-town or business districts of cities, where the streets are so frequently torn up.

Vibration and shock are exceedingly deleterious to joints which are rigid, and here again the pipe line suffers in direct proportion to its rigidity. The general results are similar to those caused by unequal settlement. Moreover, main leakage will increase as the system gets older, on account of the loosening of the joints, through contraction and expansion, settlement, and vibration.

A pipe joint in which packing materials of every character are entirely eliminated, so as to have a ground metal-to-metal connection, should give the most desirable results of all. Joints of this construction have now been in successful service for more than 15 years; and their more general adoption will proceed from the excellent results which have been obtained from their use for cast-iron mains. In joints of this type the bell and spigot ends of the pipe are machined at slightly different tapers, so that, when entered in a tight manner, the line may settle without leakage. Such a pipe line is capable of taking a rough contour over the trench bottom, or of assuming a comparatively small radius on any change of alignment.

Without the slightest leak whatever, pipe and fittings made with the machine flexible joints have been used successfully to convey hot water and fluids, such as steam, where the maximum expansion and contraction would be expected.

Furthermore, cast-iron pipe in which all jointing material is eliminated gives an ideal line. It cannot fail by subsequent deterioration of the joining material. For instance, there are many alkaline

Mr. Carson.

soils* where cement is quickly attacked by chemical action, which, however, does not corrode the pipe itself.† Moreover, such materials as lead, oakum, and cement, by virtue of their resistance to stray electric currents, are known to increase the damage to cast-iron mains, that sometimes follows the improper return of electric currents from trolley car systems.

In so far as settlement and vibration or shock become effective factors in causing leakage from cement and lead joints, or from flanged joints due to breakage in the latter, no leakage has resulted from proper metal-to-metal joints, and records have repeatedly shown that such joints have withstood successfully and without failure the most severe conditions known in practice.

Fig. 10 illustrates in a striking manner the efficiency of joints of this type. The photograph from which this half tone was made was taken in April, 1914, and shows a line that was laid across the top of a dam at the plant of the Semet-Solvay Company, at Holt, Ala. The dam was washed out by a freshet. The pipe settled as shown in the photograph, but all joints remained intact. The pipe line continued to give uninterrupted service while the dam was being rebuilt, supplying water at a pressure of about 50 lb. per sq. in. The sixteen lengths of pipe affected, of which twelve lengths were entirely without support, showed a maximum settlement or deflection of 46 in. at the center of the line.

Mr. Randlett.

F. M. Randlett,‡ Assoc. M. Am. Soc. C. E.§ (by letter).—This paper is of added interest to the writer, as the Water Department of Portland, Ore., has been experimenting with cement joints for the past year.

During 1916, there were laid 1 910 ft. of 8-in. and 362 ft. of 12-in. cast-iron pipe with cement joints, with the most satisfactory results.

For the purpose of investigation, and to illustrate the manner of using the materials, three lines of 8-in. cast-iron pipe were laid, according to the specifications of the New England Water Works Association, as follows:

Line No. 1. Neat cement joints; 3.5 lb. per joint; some waste; cement joint about 3 in. deep.

Line No. 2. Pig lead joints, caulked; 13.5 lb. per joint; 2 in. of lead.

Line No. 3. Leadite joints; 4 lb. per joint; 2 in. of leadite.

Each line consisted of eight full lengths, with the four center lengths arranged so that their supports could be removed.

* Investigations by Bureau of Standards, U. S. Dept. of Commerce (*Metal Worker*, October 6th, 1916, p. 136).

† *Technologic Paper No. 25*, Bureau of Standards, U. S. Dept. of Commerce.

‡ Portland, Ore.

§ Now M. Am. Soc. C. E.

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All were laid approximately level, and were plugged and subjected to 85 lb. internal water pressure continuously after completing and setting the joints.

The two end lengths of each line were braced securely, and were supported and weighted down with fifteen lengths of 8-in. cast-iron pipe laid across the three lines on each end.

Batter boards were set up over each of the five joints, and a center-punch mark was put on the pipe under the center line mark for each line on the batter boards. A flat space was filed on each length before being center-punched, and all measurements were taken later between the pipe and the batter boards at these points.

Several observations were taken from time to time, but Tables 3 and 4 show the results of the tests as completed. Line No. 1 was loaded with 200 lb., placed at the center of each length after the supports were removed.

Line No. 2 developed leaks at once, but not until one or two of the joints had pulled out considerably were they in such a condition that recaulking would not have been sufficient to stop the leaking entirely.

Line No. 3 developed leaks at several joints, varying from a fine stream to a slow drip. All these leaks had practically stopped at the time of the last observation.

TABLE 3.

Pipe.	Line No. 1.		Line No. 2.		Line No 3.		Remarks.
	No.	Weight, in pounds.	No.	Weight, in pounds.	No.	Weight, in pounds.	
a	2505	553	1826	550	1251	580	
b	2101	593	1227	571	1270	560	
c	2135	563	1322	566	1217	570	Suspended lengths.
d	533	565	1300	570	1329	560	
e	2439	574	1267	562	1775	582	
f	2526	566	455	560	903	577	
g	1751	570	2390	567	884	579	
h	2862	566	2220	570	1753	581	
Averages	.....	567.5	.....	564.25	.....	573.625	

Line No. 1 has developed absolutely no leaks, even under the additional load of 200 lb. per length. Lack of time has prevented loading and observing the effect of such loading to destruction of the pipe or joint. This will be done later.

Fig. 11 shows how the three mains were laid. The letters refer to the length of pipe under which they appear. The punch marks were

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lettered according to the length of pipe on which they occur, and the joints were lettered according to the length of pipe containing the punch mark. The longitudinal distance was measured from punch mark to punch mark.

TABLE 4.—Joint Tests.—Measurements on Observation Points.

The First and Last Measurement in Each Observation is on the Fixed Pipe at Each End of Each Line.

Distances, in feet, from Batter Board to Top of Pipe.

	Date: 4/24/16	Date: 5/10/16	Difference.	Distance, in feet, from point to point along center line of pipe.	Moved.	Location of punch mark.
b—1	0.855	0.942	0.097	.....	0.006 West	0.82 East of bell.
b—2	0.818	0.907	0.089	.....	0.075 "	0.77 " " "
b—3	0.781	0.825	0.034	.....	0.010 "	0.75 " " "
d—1	0.873	1.259	0.386 *b* to *d*	13.71	0.009 West	0.82 West of bell.
d—2	0.844	2.350	1.506 *b* to *d*	13.71	0.06 "	0.95 " " "
d—3	0.790	1.480	0.690 *b* to *d*	13.73	0.015 "	0.92 " " "
e 1	0.870	1.360	0.490 *d* to *e*	11.97	0.01 N. E.	0.81 West of bell.
e—2	0.830	2.988	2.158 *d* to *e*	11.975	0.015 West	0.85 " " "
e—3	0.812	1.760	0.948 *d* to *e*	11.98	0.010 East	0.75 " " "
f 1	0.802	1.160	0.358 *e* to *f*	12.285	.....	0.97 West of bell.
f 2	0.782	2.217	1.435 *e* to *f*	12.36	.....	1.155 " " "
f—3	0.770	1.401	0.631 *e* to *f*	12.45	0.015 East	1.175 " " "
g 1	0.847	0.890	0.043 *f* to *g*	12.72	0.001 West	1.62 West of bell.
g—2	0.812	0 843	0 031 *f* to *g*	12.63	.....	1.725 " " "
g—3	0.758	0.792	0.034 *f* to *g*	12.52	.....	1.62 " " "

Supports removed after first observation.

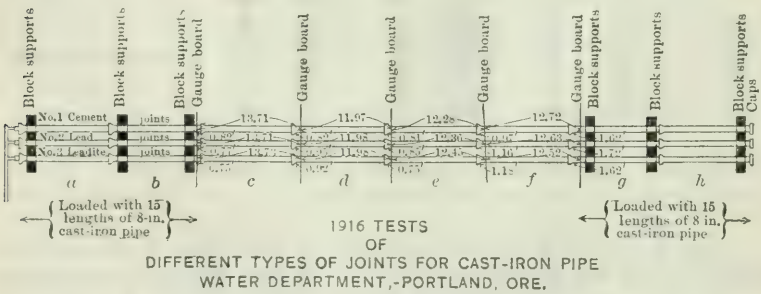

Fig. 11.

The cost per pound of the joint material was: Hemp, $0.03; lead, $0.09; leadite, $0.12; cement, $0.0055.

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The cost per joint for material was about as follows:

Line No. 1:	Hemp,	0.20 lb.	at $0.003	$0.006	
	Cement,	3.5 "	" 0.0055	0.01925	
					$0.02525
Line No. 2:	Hemp,	0.45 lb.	at $0.03	$0.0135	
	Leadite,	4.00 "	" 0.12	0.48	
					0.4935
Line No. 3:	Hemp,	0.45 lb.	at $0.03	$0.0135	
	Lead,	13.00 "	" 0.09	1.1799	
					1.1934

The average weight per foot of the 12-ft. lengths of pipe, including the bells, was:

Line No. 1: 567.5 lb. per length = 47.25 lb. per ft.
Line No. 2: 564.25 " " " = 47.0 " " "
Line No. 3: 573.625 " " " = 47.75 " " "

The actual weight of pipe suspended was:

Line No. 1: 2 288 + 800 (sand) = 3 088 lb.
Line No. 2: ..... 2 326 "
Line No. 3: ..... 2 311 "

In the case of Line No. 1, this would indicate, with the superimposed load of 200 lb. per length, an extreme fiber stress of about 9 000 lb. per sq. in. This may be within the elastic limit of cast iron, which varies from 6 000 to 20 000 lb. per sq. in.

Conclusions.—The conclusions from these experiments are that, for all ordinary mains, cement joints are superior to either lead or leadite; that leadite may often be used to advantage when time for setting of cement is not allowable; and that there are conditions where a lead joint might pull or blow out without breaking the pipe, and could be re-caulked and the main put in service more quickly than if cement were used.

The cost of making up joints is apparently in favor of leadite, with the cement joint next, and the lead joint the highest.

Mr. Shaw's statements in regard to the mixture of water and cement are substantiated by the work of Portland, except that the addition of from 15 to 20% of fine sand facilitates the ramming of the joint, and apparently does not materially weaken it. In dry weather it is necessary to keep the joint wet outside and inside, if possible, from 24 to 48 hours. If the joint is allowed to dry while setting, during the first few hours, shrinkage takes place, and this may or may not be taken up under pressure.

It is the custom of the Portland Water Department to test all lines to 50 lb. in excess of the normal pressure before putting the

Mr. Randlett.

line in service, and it may be said, for the cement joints and workmanship, that only one of those made in 1916 leaked under test.

The City of Portland has about 6 miles of 8 and 12-in. cast-iron pipe laid with leadite joints since 1912, and no leaks have ever been reported. Many of the joints sweat slightly when the pressure is first turned on, but they take up rapidly and become absolutely tight in a few days.

Most of the objection to the use of cement and leadite comes from workmen who have long been accustomed to the use of lead, but the writer has not heard of any complaint from engineers or foremen who have given either a fair trial. The Portland Water Department contemplates a very general use of cement joints in future work.

It is hoped Mr. Shaw's paper will bring out additional facts and opinions on this subject.

Mr. Pearl.

WALTER PEARL,* M. AM. SOC. C. E. (by letter).—In these days of Twentieth Century progress, and under the present duress of military requirements concerning our minerals and metals, this paper presents much matter for interesting discussion, in the way of economy, along the line of public work, as the latter must be continuous, and extend indefinitely, as long as civilization exists, regardless of whether the world is at peace or only striving for peace.

During several years of hydraulic engineering practice, embracing water-works construction for municipalities, the writer has noted the rapid exchange or substitution of the baser materials, metals, or elements, for those of refined, or combination metals and construction materials; in the case of wood, timber, and lumber, more metal is being used in the constructive combination.

Recently, bids were received in the State of Washington for a steel highway bridge, but as it was afterward found that a concrete bridge would cost only a trifle more, a bid was accepted for work of this class, considering the small margin in favor of the permanence of a concrete structure. There have been many similar instances lately, and as all branches of the Government and all individuals must now practice economy, this paper offers suggestions along many lines.

The method of making the cement joints, is described very clearly and concisely, including the method of breaking or loosening them, all of which would necessitate less time and expense, apparently, than is ordinarily required in melting, pouring, and driving lead joints, after caulking the joint with oakum or hemp in the usual manner, or the necessary expense and labor of melting the lead from the joints in disconnecting them; these elements of time and labor are favorable to the cement joint. However, more data and experience with

* Los Angeles, Cal.

the cement joint should be available before final conclusions can be reached regarding the relative cost of the two kinds of joints; former data cannot be used at the present time in making up a table showing the cost of lead joints. Mr. Pearl.

It is a question with the writer whether the cement, as described, is sufficiently wet to fill all the joint space, and, certainly, great care must be taken in driving or caulking the cement, as the joint space is limited, and irregularities in the pipe often provide very small space, even for hot lead.

Table 5, showing the joint space, etc., in cast-iron pipes, is taken from the catalogue of a manufacturer of cast-iron pipes.

TABLE 5.—Some Dimensions of Cast-Iron Water Pipe.

(Thickness of shell proportioned for 100 lb. static pressure.)

Diameter, in inches.	Length, over all.	Thickness of shell, in inches.	Depth of hub, in inches.	Joint room, in inches.
4	12 ft. 4 in.	$\frac{7}{16}$	3	$\frac{5}{16}$
6	12 " 4 "	$\frac{1}{2}$	3	$\frac{5}{16}$
8	12 " 4 "	$\frac{17}{32}$	3	$\frac{5}{16}$
10	12 " 6 "	$\frac{19}{32}$	3	$\frac{5}{16}$
12	12 " 6 "	$\frac{5}{8}$	$3\frac{1}{4}$	$\frac{5}{16}$
14	12 " 5 "	$\frac{11}{16}$	$3\frac{1}{4}$	$\frac{5}{16}$
16	12 " 5 "	$\frac{3}{4}$	$3\frac{1}{2}$	$\frac{3}{8}$
18	12 " 5 "	$\frac{25}{32}$	$3\frac{1}{2}$	$\frac{3}{8}$
20	12 " 5 "	$\frac{27}{32}$	$3\frac{1}{2}$	$\frac{3}{8}$
24	12 " 5 "	$\frac{15}{16}$	$3\frac{3}{4}$	$\frac{3}{8}$

The figures in Table 5 may be assumed as standard, but as the flask, mould, and pattern vary slightly with different manufacturers, the "joint room" may suffer. Table 5 shows what a limited space there is in the hub or bell for cement after the spigot end is entered, for a thickness of not less than $\frac{1}{4}$ to $\frac{1}{2}$ in. of cement should be required throughout the entire joint. This thickness might prevent the possibilities of seepage or leaks in the joints of the pipe constructed under the author's supervision. The paper gives no information regarding the joint space. In laying a line of pipe such as described, great care would be necessary in securing, as nearly as possible, perfect alignment and grade, in order to have uniform joint space.

It would seem that cement joints could only be used with straight lines of pipe under low pressures. In a joint of a 6-in. cast-iron pipe, when laid straight, the space is $\frac{5}{16}$ in., and on a curve of 250 ft. radius, the resulting space would be practically $\frac{3}{16}$ in., or $\frac{1}{8}$ in. on a curve of 166 ft. radius. Numerous ups and downs in grade could not be permitted, thus showing the necessity of laying such pipe according to the method of laying sewer pipe. This matter of joint space has been gone into at some length by the writer, as he realizes

Mr. Pearl. the difficulty of pouring the lead in a close joint, and more especially in caulking it.

Another question arises, as to whether cement joints in a cast-iron main will stand the strain and shocks due to the sudden closing of valves and hydrants (though not necessary, such things sometimes happen), causing water-hammer, ram, and vibration in the line of pipe, which might crack or injure the cement joints, causing leakage, expense, and annoyance. This is likely to occur in a gravity pipe line, and, in a pumping line, the constant vibration might cause seepage and leaks to develop in the cement in due time. It is well known that cement and concrete will crack and disintegrate where there are sudden shocks or continuous vibration, though it will stand great pressures due to head or weight, if not subject to disturbance.

The author has described cases where pipes with cement joints were laid in made or filled ground, and where settlement occurred without injuring the joints or causing leakage; also where a parallel trench caused the caving of the pipe trench, leaving the pipe hanging unsupported in the air for a time, without even causing seepage in any of the joints. The reason for this, doubtless, was the gradual giving way of the ground supporting the main. Had the shock been sudden, the results might have been different.

It may be conceded that no other base metal having the qualities of lead—so ductile and homogeneous—seems to be as well adapted for the joints of cast-iron pipe under pressure. Lead joints may be driven or re-caulked, and all leakage stopped, while the pressure is on, which is quite an advantage over cement, as it would be impossible, practically, to secure any bond with the cement after it had thoroughly set. In case of an important leak it would be necessary to shut off all pressure on the main, and possibly the water would have to be drawn off; then, after removing all the original cement, an entire new joint would have to be made.

In a grade-line pipe, there is no doubt that cement joints would be safe and economical; but in a high-pressure cast-iron main, it would appear to the writer to be difficult to find a substitute for lead joints; under the present, prevailing high prices of materials, and especially metals, the cost and the special requirements relative to the safety of the structure, should be well considered before work begins.

It is possible that this paper may bring out some discussion relative to the likelihood of electrolysis along a cast-iron main with cement joints paralleling electric car lines; it would seem that, as cement is a non-conductor, such a line of pipe would be unfavorable to electrolytic action, as the current being so frequently broken would leave

the main in minor quantities without causing deterioration; however, experiments would demonstrate this matter more satisfactorily. Mr. Pearl.

The author states that the cement joint for cast-iron pipe has passed the experimental stage, especially in work with which he has been connected, and that such joints are safe and satisfactory. The work seems novel to the writer, and he is indebted to the author for attracting his attention to new construction methods.

H. G. Moulton,* M. Am. Soc. C. E.—With respect to the use of Portland cement instead of lead in forming joints for cast-iron water mains, it may be said that this is the method against which there is every theoretical objection, but in favor of which there is the practical argument that it has become a demonstrated success in actual practice. For a number of years past the use of cement for this purpose has been standard practice in Los Angeles, a thriving municipality of some 350 000 population, and the speaker is indebted to William Mulholland, M. Am. Soc. C. E., Chief Engineer of the Bureau of Water Works and Supply of that city, for much interesting information in regard to this method of making joints. The following statements in regard to costs and conditions in Los Angeles are in a large measure based on figures furnished by Mr. Mulholland. Mr. Moulton.

The advantage in favor of this method lies in its great economy. In 1912 about 9 000 ft. of 30-in. high-pressure water main were laid in Los Angeles. Cement at that time was quoted at $2.00 per bbl. there and lead cost 5 cents per lb. The total saving on the job by using cement instead of lead was approximately $3 500. At present comparative prices of lead, cement, and labor, the saving would be very much greater, and, of course, on 36 or 48-in. mains a very large saving is possible.

An additional advantage in the use of cement lies in its insulating effect, in that it appears to act as a perfect seal between the separate sections of pipe and thus to reduce materially the effect of electrolysis. As damage to cast-iron pipe from stray electric currents has resulted in deterioration of water mains in many places, any type of joint which tends to reduce such damage, by stopping the flow of stray currents along the pipe, is worthy of serious consideration.

The arguments against the use of the cement joint are based on a fear that temperature changes, resulting in uneven expansion and contraction, would tend to break up the joint, and also on doubt as to the action of pipe caulked by this method in the event of settlement, in filled ground or otherwise. In Los Angeles, the usual practice is to refrain from the use of cement joints where pipe must be laid on fills, probably on the assumption that, in case of settlement opening up joints, they can be re-caulked more easily if lead is used. In Long

* New York City.

Mr. Moulton.

Beach, however, as mentioned in the paper, the cement joint has been used in filled ground, and also under conditions where trenches adjacent to the pipe have allowed sections of it to sag over a length of 40 ft. in one instance; and another instance is mentioned in the paper where some 98 ft. of pipe broke away and dropped into a trench with all the joints remaining in perfect condition, except those at the actual point of rupture.

In regard to the question of the ability of pipe to span extensive distances when the supporting ground is washed away, it may be said that, in designing connections for water pipe lines, good practice does not call for laying them out as suspension bridge connections, under the assumption that the pipe should be able to hang suspended over extensive spaces without failure. Under all normal conditions, a water pipe joint need only be considered in regard to its ability to prevent the leakage of water with reasonably continuous support and with a proper depth of cover above. Its behavior under abnormal conditions, where it has to hang suspended over wash-outs or cave-ins, is a matter of interest only from a standpoint of curiosity, and one is certainly not justified in designing all pipe on the assumption that it must meet conditions such as this, and providing special connections for this purpose.

In regard to the question of temperature changes, it may be said that there is seldom a variation of more than 50° between the winter and summer temperatures in city water mains. In winter, temperatures lower than 32° Fahr. cannot exist, on account of the fact that, at this point, the water changes to ice; and above a temperature of 85°, it is certainly too hot for use as drinking water. In Los Angeles the maximum variation in temperature is from 45° Fahr. in winter to 82° in summer, or a total range of 37° Fahr. Under this range, no trouble has been experienced with cement joints from leakage introduced by temperature stresses.

This paper brings up an interesting method which was first devised in California, and the success of which has been proved there on an extensive scale. It is no longer in an experimental stage, and is worthy of serious consideration on the part of eastern municipalities. The saving in expense made possible by the substitution of cement for lead in the joints of cast-iron pipe is so great that all the larger eastern cities would be justified in commencing immediately the use of cement joints in an experimental way in outlying districts, working gradually in to more important parts of the water system as the advantages and limitations of the method are developed in each municipality.

The author is entitled to a large measure of credit for having brought thus forcibly before the Society the advantages of a pioneer engineering method having great possibilities, which the Engineering Profession as a whole has been slow to recognize and adopt.

H. B. Lynch,* Esq. (by letter).—This method of caulking joints in cast-iron pipe deserves a wider use, not only on account of its low cost, but because it makes a better joint, at least where no considerable movement of the pipe is expected after it is laid. Mr. Lynch.

Cement joints have been adopted as standard at Glendale, Cal., and have been used on 10 miles of cast-iron pipe laid in the past 2 years. This joint was not adopted until investigation had shown that it is being used with perfect satisfaction in various Southern California localities, and tests had been made to show its effectiveness. The results obtained at Glendale have been similar to those described in the paper. No joint yet placed has shown permanent leakage, and, in only four, has the presence of dampness been detected after the pressure was turned on. No joint has required recaulking.

The strength of this joint has been shown at Glendale by several incidents of various kinds. As a matter of fact, it is customary for the pipe crew to take the greatest liberty with the pipe after the joints are completed, as experience has shown that a joint properly made will not be started by quite severe treatment. A 3-mile pumping main of 20-in. cast-iron pipe recently completed was entirely laid up with cement, at an estimated saving of more than $3 000. On this line, when a nipple was to be caulked into a cross, it was customary for the men to stand the cross on end on the surface of the ground and set the nipple into the top bell, and caulk up on the surface. The nipple and cross, weighing 3 000 lb. were then picked up with the chain blocks and lowered into the trench, without waiting for the set to occur. No joint treated in this way showed the slightest leakage or dampness under a pressure of 90 lb. On one occasion, a 20-in. cross had caulked into it a plug, a reducer, and a 20-in. nipple, 10 ft. long. This whole assembly, weighing about 4 000 lb., was lowered into place as soon as caulked, and is at present operating under a pressure of 90 lb. per sq. in. without seepage.

This 20-in. line was connected to a 16-in. riveted steel line. For this purpose the 16-in. line was taken out of service at 8 p. m., and put back at 4.30 a. m. Joints were given 4½ hours to set. Out of eight joints thus treated, one remained damp for one day. One joint, where the 16-in. riveted line was caulked into a cast-iron bell, dripped for two days and then took up tight. This joint showed a face of about 1¼ in. between the pipe and the bell. No other joint on this job showed any initial seepage, and all are tight now.

The method of making a joint in Glendale is much the same as that described in the paper. After yarning, the joint is rammed full of slightly damp cement. Great care is taken to caulk this first ring of cement thoroughly, as this is the greatest factor in a successful joint.

* Mgr., Public Service Dept., Glendale, Cal.

Mr. Lynch. No bead is now used, as no difference in results is noted when this is omitted. After caulking, the joints are covered with earth for protection against the sun.

The tools are slightly wider than those used for caulking with lead.

In spite of the strength of the joint, its removal is surprisingly easy. At different times, 20-in. plugs have been taken out, and there has not been the slightest difficulty where the method described in the paper was used. To caulk a joint of this size the writer allows about one hour's time of one man.

Mr. Bowen. Edward R. Bowen,* Assoc. M. Am. Soc. C. E.† (by letter).—The writer has read this paper with keen interest. It gives in admirable detail the results of an extended experience in handling the cement joint in cast-iron pipe construction. The advantages of this type of joint have not been generally realized among water-works engineers. It is not only less expensive than the lead joint, but, in some ways, distinctly better.

As protection against electrolysis it is almost a perfect insulator. Tests made at the Long Beach Water Department yards showed the resistance of cement joints on an 8-in. line to be nearly 19 ohms, or more than three times the resistance of the ordinary lead joint. Because of the high resistance, under ordinary electrolytic conditions, extremely small currents could travel along the pipe line. Tests were made in Long Beach in the vicinity of an electric railway sub-station to determine the quantity of current carried by the water-pipe lines. In this particular location there were cast-iron lines with both lead and cement joints. The position of the pipe lines with respect to their susceptibility to electrolytic action was practically the same. An appreciable flow of current was discovered in the pipe lines with the lead joints, and a very much smaller flow was noted in the line with the cement joints. In the latter instance, however, the same drop in potential could be obtained by inserting the connections in the soil at the same distance apart as they were applied to the pipe lines.

In the writer's opinion, the success or failure of the cement joint depends solely on the method used in its construction. It is extremely important to have as little moisture in the cement as possible. The mixture described in the paper has proved to be entirely satisfactory, and care should be taken to avoid using more moisture than is there recommended. If more water is used the cement in setting shrinks away from the pipe, resulting in a leaky connection.

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* Los Angeles, Cal.

† Now M. Am. Soc. C. E.

Tables 6 and 7 have been prepared to illustrate the relative costs of the two types of joints. Table 6 is based on the data in the paper. Table 7 is based on the cost data assembled by the Los Angeles City Water Department. Mr. Bowen.

TABLE 6.—Cost of Cement Joints.

Diameter of pipe, in inches.	Jute.		Cement.		Labor.		Total cost of joint.
	Pounds per joint.	Cost at 6 cents per pound.	No. of joints per sack.	Cost at $1.00 per sack.	No. of joints per 8-hour day.	Cost at $2.75 per day.	
4	0.14	$0.008	24	$0.042	50	$0.055	$0.105
6	0.19	0.011	18	0.056	42	0.065	0.132
8	0.24	0.014	14	0.071	34	0.081	0.167
10	0.43	0.026	11	0.091	28	0.098	0.215
12	0.51	0.031	8	0.125	24	0.115	0.270
14	0.58	0.035	7	0.143	20	0.138	0.315
16	0.66	0.040	6	0.167	17	0.162	0.368
18	0.73	0.044	5	0.200	14	0.196	0.440
20	0.80	0.048	4	0.250	11	0.250	0.548
24	0.95	0.057	3	0.333	7	0.393	0.783

TABLE 7.—Cost of Lead Joints.

Diameter of pipe, in inches.	Lead.		Yarn.		Fuel at 0.6 cent per pound of lead.	Labor cost, based on $2.75 per man per day.	Total cost of joint.
	Pounds per joint.	Cost at 6 cents per pound	Pounds per joint.	Cost at 6 cents per pound.			
4	7	$0.420	0.2	$0.012	$0.042	$0.110	$0.584
6	9	0.540	0.4	0.024	0.054	0.168	0.786
8	14	0.840	0.5	0.030	0.084	0.216	1.170
10	16	0.960	0.6	0.036	0.096	0.230	1.322
12	20	1.200	0.8	0.048	0.120	0.264	1.632
14	24	1.440	0.85	0.051	0.144	0.310	1.945
16	29	1.740	1.0	0.060	0.174	0.324	2.298
18	31	1.860	1.1	0.066	0.186	0.360	2.472
20	35	2.100	1.3	0.078	0.210	0.432	2.820
24	40	2.400	1.5	0.090	0.240	0.492	3.222

George W. Pracy,* Assoc. M. Am. Soc. C. E. (by letter).—Several months ago the Spring Valley Water Company laid 4 730 ft. of 4-in., 900 ft. of 6-in., and 827 ft. of 8-in. cast-iron bell-and-spigot pipe, with cement joints. Mr. Pracy.

The cement was mixed by one man who handed it around to the two or three men making the joints. No caulking was done on the

* San Francisco, Cal.

Mr. Pracy.

joints, the cement being tamped by hand with a caulking iron. After the joint was full, a bead was put on around the face of the bell. Neat cement was used. Just as little water as possible was added in mixing. When ready for use, the wet cement was still dry enough to crumble when handled. The first few joints were filled with a wet mix, with the result that, in these joints, the cement shrunk away from the iron, and all the joints had to be remade. As an experiment, one of the 8-in. joints was caulked hard, using an extra dry cement. This gave a good joint, but not better than the others, and it took twice as long to make.

Water was turned into the pipes 48 hours after the last joint was made. For the first day nearly every joint leaked. After that, they took up rapidly and, at the end of 1 week, all were tight.

The pipe was laid in five sections of about 1 300 ft. each. All but one section was tested by measuring the quantity pumped into the section during a given period. Tests were made after the pipe had been under ordinary working pressure for about 1 week to 10 days. For the first section laid, this leakage was $\frac{1}{3}$ gal. per lin. ft. of pipe joint per 24 hours. The second leaked so slowly that the leakage could not be measured. The last two sections laid were absolutely tight. The lines were pumped to pressure and left standing, in one case, for $\frac{1}{2}$ hour and in the other case for $2\frac{1}{2}$ hours, without any drop in pressure. The 6-in. pipe section could not be tested, as it was necessary to put it into use immediately.

The quantity of cement used was as follows:

No. of joints.	Size.	Quantity of cement, in pounds.
187	4-in.	750
2	5-in.	
12	8-in.	
78	6-in.	360
168	4-in.	1 715
86	8-in.	

These figures include all waste. On an average the 4-in. joints took about 4 lb., the 6-in. joints 5 lb., and the 8-in. joints from 7 to 8 lb. These figures are rough approximations, based on the theoretical quantity necessary for a joint of each size.

The fire hydrants on the line were set with lead joints, as it was feared that the vibration of the hydrants, when in use, would break the cement joints.

The lines have given entire satisfaction.

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Paper No. 1430

CONSTRUCTION METHODS USED IN BUILDING THE LOWER RESERVOIR DAM OF THE BALMORHEA PROJECT*

By Vernon L. Sullivan, M. Am. Soc. C. E.

With Discussion by Vernon L. Sullivan, M. Am. Soc. C. E.

Synopsis.

The object in writing this paper is to present economical and practical methods of constructing earth dams. In many cases, where the foundations are poor and a long section is to be built, these methods may suggest a solution which will enable the construction of a dam which, otherwise, would be impracticable or expensive.

The Balmorhea Project formerly consisted of a diversion canal which distributed the water of some very large springs for irrigation purposes. As the supply was insufficient for the area desired to be irrigated, there was constructed a reinforced concrete diversion dam, 500 ft. long, across a large and wide canyon; an intake canal, 4½ miles long, with a carrying capacity of 1 000 cu. ft. per sec.; a large earth dam, 3 900 ft. long, with a maximum height of 47 ft. above the lowest point in the creek bed, and having reinforced concrete paving on the

* Presented at the meeting of May 15th, 1918.

inner slope; an outlet conduit, well, and controlling gates; and an outlet canal, 2½ miles long.

Most of the excavation for the dam was done with a drag-line dredge, and the earth embankment was put in with twenty-six dump-wagons loaded by two elevators. The principal features of interest in this work were the excavation for the cut-off or puddle trench under the dam, the methods of settling and compacting the earth embankment, and the manner of providing a more or less flexible and expansible concrete pavement on the inner slope.

A small supply ditch, about 3 miles long, was first constructed, up the creek from a point on the hill at the height of the top of the proposed dam, in order to divert and provide water for construction purposes, principally for puddling the clay cut-off or core-wall, irrigating the earth lifts, and for concrete.

The cut-off trench, along the axis of the dam, was excavated through a rocky and gravelly surface and down to the rock or clay sub-surface, running from 0 to 20 ft. deep and from 10 to 40 ft. wide. This work (Fig. 1) was practically all done with a drag-line excavator, the excavated material of the better kind being used in the lower section of the dam.

As soon as sufficient excavation had been completed, a clay dam was placed across the trench, and the latter was filled with water from the supply ditch. Then selected clay was bull-dozed into the water-filled trench, as illustrated by Fig. 2. The filling of this trench through the water—shoving it in from the end all the time—made a complete puddle of the clay without any additional mixing.

The remaining portion of the natural surface under the proposed dam was cleared and thoroughly plowed, and then the earth embankment was put in, in lifts 3 or 4 ft. thick. Each lift was then irrigated thoroughly, as illustrated by Figs. 3 and 4, by bordering, checking, and flooding with water. The water put on each lift was sufficient to cause the moisture to unite thoroughly with the moisture of the previous lift, tests being made with post-hole diggers before the next lift was put on, and, when necessary, a second flooding was made, the object being to obtain a complete and uniform settlement of the earth, in the most economical and practical manner possible. As this method of actually saturating the earth with water quickly effects a most complete settle-

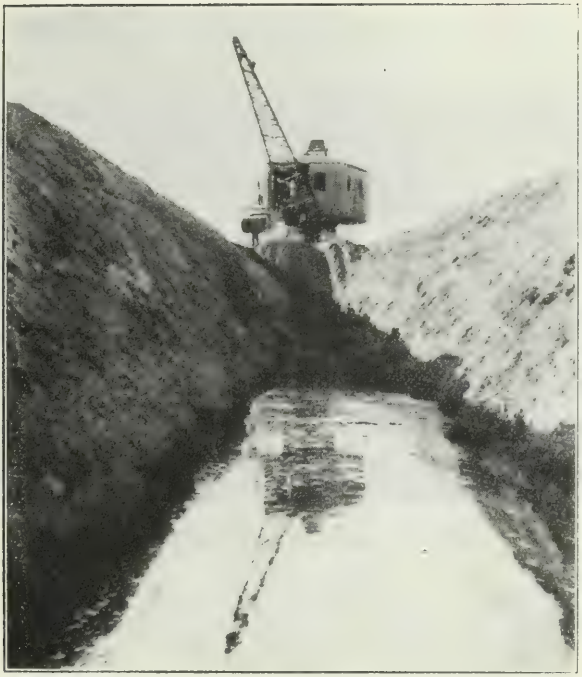

Fig. 1.—Excavating Puddle Trench.

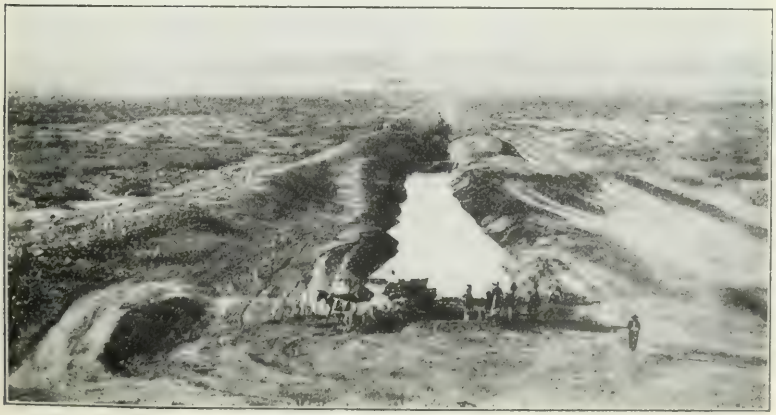

Fig. 2.—Puddling Trench.

Fig. 3.—Balmorhea Dam, During Construction.

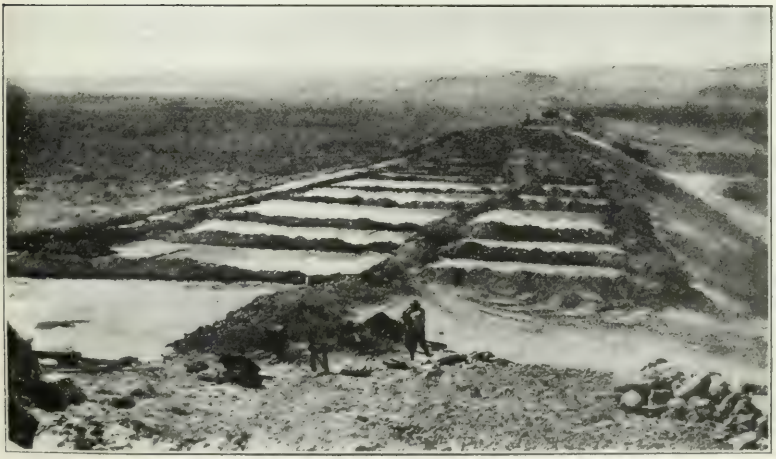

Fig. 4.—Irrigating Each Lift of Dam.

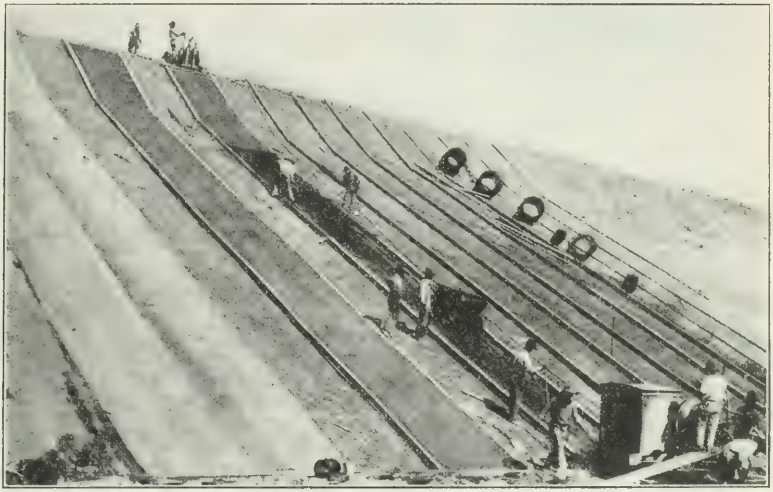

FIG. 5.—PAVING BALMORHEA DAM WITH REINFORCED CONCRETE.

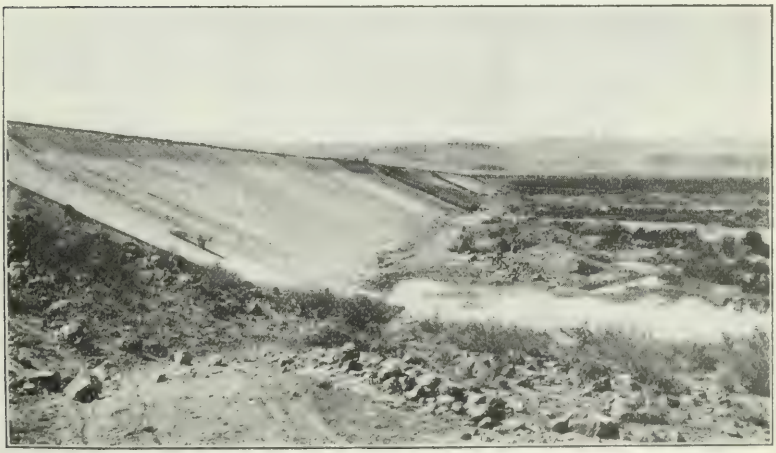

FIG. 6.—MAIN STORAGE DAM, BALMORHEA, TEXAS.

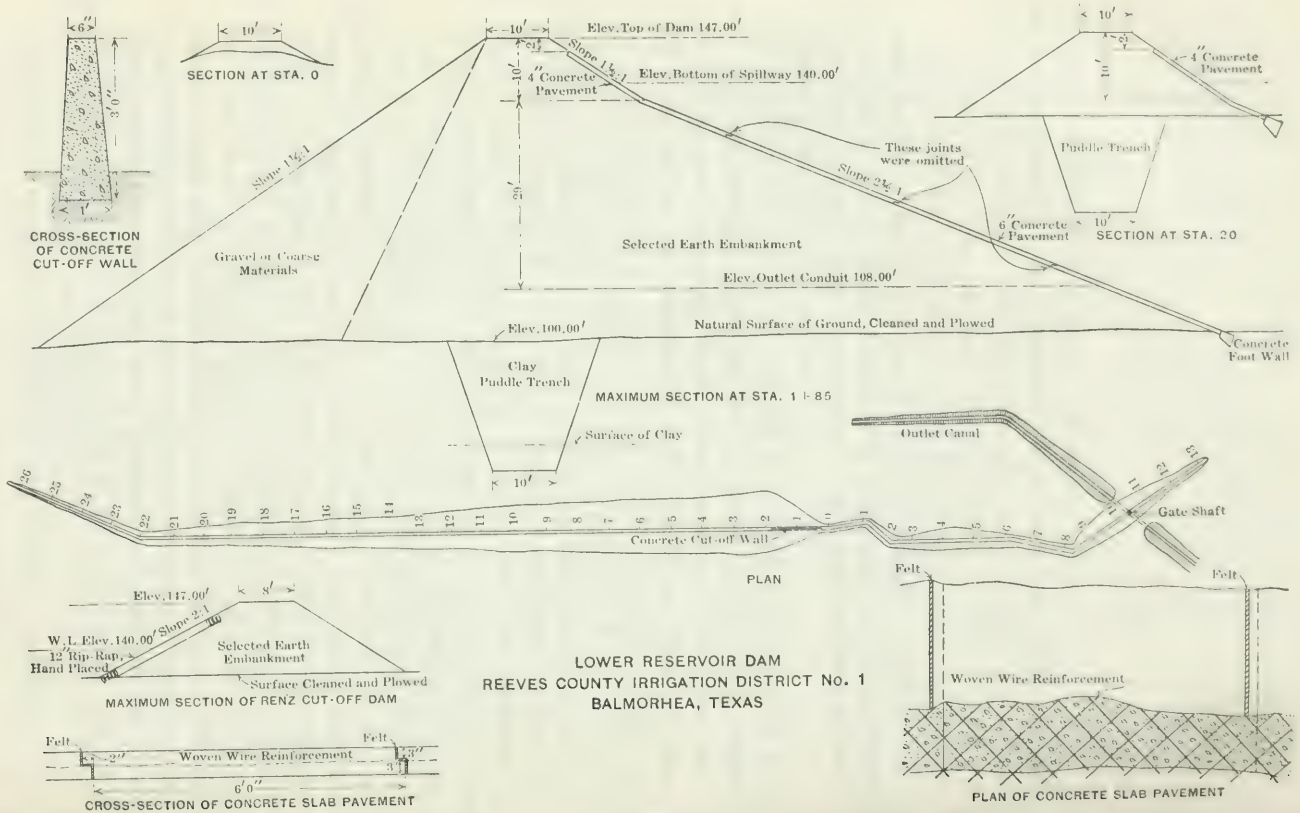
6″
3′ 0″
1′
CROSS-SECTION OF CONCRETE CUT-OFF WALL
10′
SECTION AT STA. 0
Elev. Top of Dam 147.00′
10′
Slope 1½:1
Elev. Bottom of Spillway 140.00′
4″ Concrete Pavement
Slope 1½:1
These joints were omitted
Slope 2½:1
20′
Gravel or Coarse Materials
Selected Earth Embankment
Elev. Outlet Conduit 108.00′
Elev. 100.00′
Natural Surface of Ground, Cleaned and Plowed
Concrete Foot Wall
Clay Puddle Trench
MAXIMUM SECTION AT STA. 1 + 85
Surface of Clay
10′
4″ Concrete Pavement
Puddle Trench
6″ Concrete Pavement
10′
SECTION AT STA. 20
Outlet Canal
Gate Shaft
Concrete Cut-off Wall
PLAN
Elev. 147.00′
8′
W. L. Elev. 140.00′
Slope 2:1
12″ Rip-Rap, Hand Placed
Selected Earth Embankment
Surface Cleaned and Plowed
MAXIMUM SECTION OF RENZ CUT-OFF DAM
Felt
2″
Woven Wire Reinforcement
Felt
3″
6′ 0″
CROSS-SECTION OF CONCRETE SLAB PAVEMENT
Felt
Felt
Woven Wire Reinforcement
PLAN OF CONCRETE SLAB PAVEMENT
LOWER RESERVOIR DAM
REEVES COUNTY IRRIGATION DISTRICT No. 1
BALMORHEA, TEXAS

FIG. 7.

ment, and has been found to produce an impervious embankment, it is particularly suited to dam construction.

The spilling point of the reservoir is at Elevation 140, the reinforced concrete paving extends to Elevation 145, and the top of the earth dam when completed will be at Elevation 147. The front or inner surface of the dam is protected from wave action and boring animals by a more or less flexible and practically impervious reinforced concrete paving, as shown by Figs. 5 and 6. A toe-wall, or foot-wall, was built along the inner toe of the dam, to provide a footing for the concrete paving. This paving was constructed in slabs, 6 ft. wide, extending from the toe-wall to within 2 ft. of the top of the dam. It is 4 in. thick at the top, or Elevation 145, and increases in thickness uniformly to 7 in. at Elevation 100. The slabs were put in alternately, with the offset or step joint, 2 in. wide, running their entire length. The entire surface was reinforced by running heavy wire netting, of about 1 ft. mesh, diagonally across the slabs, and crossing the joints at the same elevation as the step. A thin felt strip, laid over the wire and up the sides of the step joints, provided a parting point between the slabs for expansion. The wires running diagonally through this step joint will provide sufficient give and take in the 2-in. horizontal part of the step for expansion and contraction, without undue strain on the reinforcement. Thus the entire surface of the paving is completely reinforced and bound together in a manner which gives flexibility and provides for expansion.

The dam is built on the least section compatible with good engineering. The back slope is $1\frac{1}{2}:1$; the front slope is $2\frac{1}{2}:1$ from Elevation 100 to Elevation 138, and $1\frac{1}{2}:1$ from Elevation 138 to the top.

The outlet was cut through a low saddle in a string of hills surrounding the reservoir, and from it there is built a reinforced concrete conduit, with double wells, and high-pressure emergency and service gates of standard design. The section and general plan of dam, together with the paving and other details, are shown on Fig. 7.

DISCUSSION

Mr. Sullivan.

VERNON L. SULLIVAN,* M. AM. SOC. C. E. (by letter).—Since the paper was written, the Balmorhea Dam has been put to a severe test. Early in March, 1919, a hard wind swept down the draw at practically right angles to the dam, the waves hitting the dam squarely. The storm at times almost reached the proportions of a hurricane, and was so severe that men could not stand on the dam.

It had been planned, when the earth had had time to settle, to put a vertical 2-ft. coping wall, with an overlying cap, on top, to bring the concrete slab rip-rap up to the crest of the earth dam, so as to turn back any waves that might reach that height. This had not been done when the storm occurred. The water in the reservoir was at an elevation of 137.5 ft., or ½ ft. above the change in the slope from a 2½ to 1 to a 1½ to 1, and 9½ ft. below the crest of the dam.

During the storm the waves rolled up the concrete slab rip-rap to such a height that the wind would catch under them and shoot them up the slab and over the dam; many of the waves never touched the earth dam at all, the water hitting the ground on the back side of the dam below the lower toe.

This continual action for 36 hours washed down the earth at the back of the earth dam to a point where it had been settled and packed by irrigation process during construction.

On the front of the dam, the water got behind the concrete slab rip-rap and broke it down to an elevation of 137 ft., for a distance of nearly 1 000 ft., at a point where the wind and wave action was most severe. Very little damage was done below the point of the change of slope in the concrete slab rip-rap.

Two lessons were learned: First, that a concrete reinforced slab rip-rap is good protection for earth dams where the slope is 2½ to 1 or flatter; and, second, that on a concrete slab rip-rap the surface should be roughened by setting rocks in the concrete so as to project above the surface line to break up the waves.

* El Paso, Tex.

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Paper No. 1431

IMPROVING ARCH ACTION IN ARCH DAMS*

By L. R. Jorgensen, M. Am. Soc. C. E.

With Discussion by Messrs. W. P. Creager, S. H. Woodard, W. H. R. Nimmo, and L. R. Jorgensen.

Synopsis.

In this paper the reasons for arch (rib) shortening, and the resulting opening up of the contraction joints on unloaded arch dams, are first discussed.

Crown deflections, plotted from measurements taken during a series of years on an arch dam, are given and discussed. The deflection curves indicate very plainly the great influence of temperature changes on the deformation of arch dams, and therefore, also, on the stresses in the building material (concrete) used.

It is shown how the practical arch deviates from the intended or theoretical arch; and, finally, a method is outlined for pressure grouting the spaces between adjacent walls of the contraction joints along the arch, whereby a key-stone, so to speak, is inserted under pressure into each contraction crack. This grouting is intended to be done at the end of a cold season, some time after the dam has been completed, and with the reservoir empty, or as nearly empty as possible.

* Presented at the meeting of June 5th, 1918.

The grouting puts initial compression into the arch, whereby the shearing and cantilever stresses can be cut down, perhaps to half the value they otherwise would have.

The water-tightness of the structure is also improved by this grouting operation.

Introduction.

The writer has been connected with arch dam design and construction for some years, and has taken particular interest in watching the behavior of several existing dams, especially of the arch type.

The fact that arch dams, and mainly those of heavy sectional area, do not develop arch action before considerable load has been applied to them is generally known, but, to a great extent, has not been especially considered in their design.

At present, working stresses of about 25 tons per sq. ft., as used in ordinary dam practice, are sufficiently low to produce a safe structure, even in spite of the fact that the formulas used for the design do not consider everything.

It is obvious, however, that if the practical arch could be constructed in such a way as to coincide more nearly with the ideal arch—that is, one which acts as an arch from the very start, and in which the load is mainly carried by pure arch action—a still safer structure would result. Higher unit stresses than are now common could be used with safety, if arch action could be made more definite from the start. The factor of safety to be used would depend on the importance of the particular structure in each case.

The two main reasons for the phenomenon that an arch dam does not act as an arch before considerable load has been thrown on it are, as is well known: First, the shortening of the arch rib due to shrinkage; and, second, because the arch is fastened to the foundation.

The shortening of the arch rib is caused by the natural shrinkage of the concrete as it sets, the dissipation of the chemical heat, the permanent set after the reservoir has filled once, and the fact that dams are of necessity almost always built during the season of highest temperature.

This shortening of the arch rib is plainly visible on dams provided with contraction joints. With an empty reservoir, during cold weather,

these joints will have opened up from ¼ in. to perhaps as much as ⅜ in., where spaced, say, 60 ft. apart. If not provided with contraction joints, a dam of heavy cross-sectional area will undoubtedly show cracks at more or less regular intervals from 40 to 80 ft. apart. A dam of slim cross-sectional area may not always crack, as it is somewhat flexible, and the crown can often be pulled in a down-stream direction without developing excessive tension in the arch.

In cases where the contraction joints have opened up, the water level in the reservoir will have to rise a certain distance in order to throw sufficient load on the structure to force the dam, acting as a cantilever, to deflect in a down-stream direction, before the voussoirs—that is, the bodies between the contraction joints—come into contact with one another. Only from that moment does the remainder of the water load divide up between the arch, the curved beam, and the cantilever.

In the case where the arch has not cracked, although under the influence of tensile stresses, the water level in the reservoir will have to rise until this tension has been compensated for by pressing back the cantilever. The remainder of the water load is then divided up between the arch, the curved beam, and the cantilever, according to their relative capacities for sustaining load.

The fact that the arch is fastened to the foundation prevents any considerable arch action from taking place at or near the latter; this, however, is not detrimental to the safety of the structure, such as would be the shortening of the arch due to shrinkage. The load on the lower portion of the arch causes shearing stresses in the concrete next to the rock and for some distance above and below. These shearing stresses, in turn, cause an elastic deformation, of both the dam and the rock foundation, to take place in a down-stream direction, especially at the crown of the arch, where the movement in that direction is the greatest. Ordinarily, the rock bottom has a higher modulus of elasticity than the concrete; it is well supported laterally from all sides, and therefore offers greater resistance to this movement (perhaps three times) than the concrete, but there is nothing to warrant the assumption that the rock bottom is immovable, and measurements also seem to indicate that it is not, although measurements of such movements are very difficult to make correctly, as they are so small in this region.

A trial calculation will ordinarily indicate very low unit shearing stresses at the foundation, even assuming that shear alone carried all the load on, say, the lower fifth of the dam.

The unit shearing stresses, in most cases (perhaps in all cases), will be less than the unit weight multiplied by the coefficient of friction; but, in any case, the weight of the structure will exert considerable unit compression perpendicular to the horizontal plane of shear, and thereby improve the ability of the concrete to withstand shearing stresses along horizontal planes.

Deflection Curves.

The curves shown on Figs. 1 and 2 are typical for the deflection of the crown of an arch dam due to different water loads and different temperatures of the dam body. The curves are plotted from measurements taken on the Salmon Creek Dam, near Juneau, Alaska. This dam, Fig. 4, is of the constant-angle arch type, 168 ft. high, with a crest span of 550 ft.*

Fig. 3 shows the maximum cross-section at the crown. On this cross-section is indicated a number of horizontal steel rods embedded in the masonry, 20 ft. apart. These steel rods are provided with copper points on the down-stream end for weather-proofing purposes. The different readings were obtained by sighting these points through a transit and determining their position in regard to fixed bench-marks.

It is very difficult to measure the deformation close to, or at, the foundation. It is so small that a transit reading would not be accurate enough, and if a stationary measuring apparatus was arranged on the ground, down stream from the dam, it would have to be at least 50 ft. distant in order to be on ground which does not take part in any movement caused by the load on the dam. The measuring tape or rod then becomes so long that all measurements taken must be compensated for temperature changes, and even a small error in such calculations or estimates will be sufficient to make this method of doubtful value. The best thing to do seems to be to extend downward, in a uniform direction, the curve obtained from observations at higher elevations.

The measurements on the Salmon Creek Dam were commenced on October 8th, 1914, when the reservoir was filling for the first time.

* A complete description of this dam may be found in *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 702.

and the position of all the copper points on that date is taken as zero for all curves shown on Figs. 1 and 2. The actual zero will lie a little to the left; it may be some time before the water will be low enough to determine it by measurements.

The load due to the water level shown for Curve No. 1 was sufficient to keep the contraction joints (only two) closed.

There is not much that need be explained about the results of the first and second deflection measurements—those taken on October 26th, and November 24th, 1914. The water kept rising, deflecting the crown in a down-stream direction, about as might have been expected. The fourth measurement, Curve No. 4, taken on May 18th, 1915, begins to show some peculiarity about this dam, that is, the knee in the deflection curve at Elevation 1095, which is still more apparent on the three following curves. This peculiarity is caused by construction features which must be known to be appreciated.

As it was necessary to stop construction work late in the fall of 1913 at about this elevation, the zone of the dam in this vicinity was built during the coldest portion of the fall and the coldest portion of the following spring. The total shrinkage of the concrete in this zone, therefore, has been less than the average, and the arch, therefore, takes a greater proportion of the total load than it does either above or below; in fact, some load is transferred through the vertical beam (the cantilever) to this zone from both above and below. Such transference of load also takes place through the vertical beam at the crest, as shown by Curve No. 4, but this would be expected in any case.

Curve No. 5 shows considerably more excess deformation of the dam than Curve No. 4, although the water load is only slightly higher, but Curve No. 5 was plotted from measurements taken on December 2d, 1915, at a time when the days had been short and cold for some time, compared with the long, warm days around May 18th (Curve No. 4). Curve No. 5 also indicates that the outside temperature has been lower than that of the water and the dam body below the water level for some time, as the portion of the dam above the water level is forced down stream to a greater extent than the lower loaded portion as a result of the greater shortening of the arch (rib) in the upper region, produced by a lower temperature in the upper exposed portion of the dam.

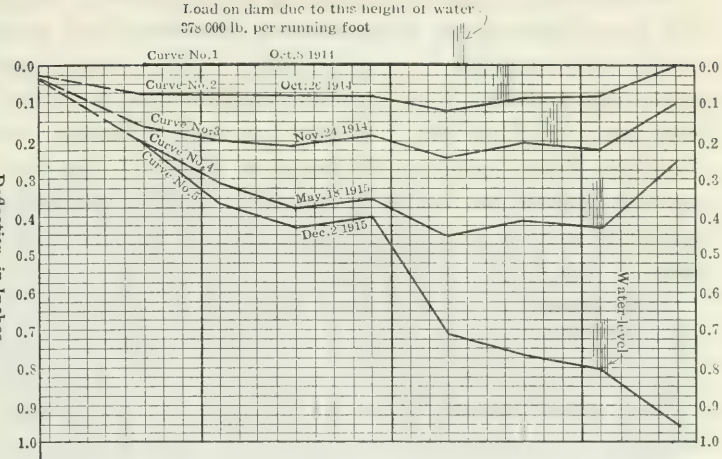


FIG. 1. DEFLECTIONS OF SALMON CREEK DAM.

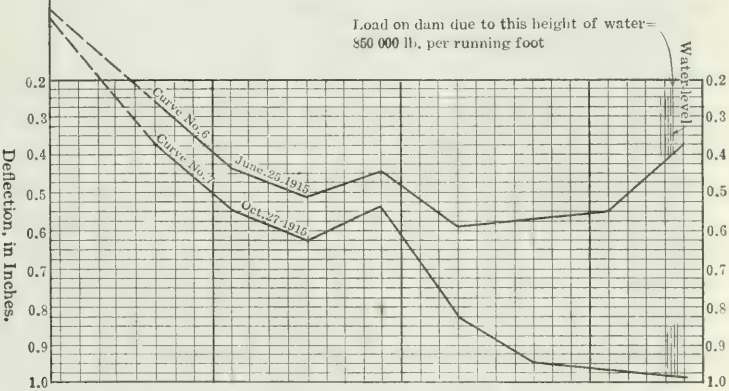


FIG. 2.

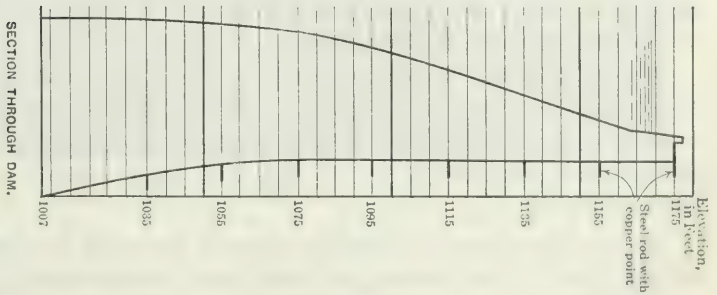


FIG. 3.

Curves Nos. 6 and 7 give an idea of the magnitude of the deformation due to temperature changes alone. The water load is the same in both cases—that due to reservoir full—but the average temperature of the dam body was high when the measurements for Curve No. 6 were taken, on June 25th, 1915, and low when the measurements for Curve No. 7 were taken, on October 27th, 1917. The days, of course, are still shorter and colder in January (Juneau, Alaska) than in the latter part of October, but October 27th was the last day the reservoir was full to the spillway crest.

Both curves are of the shape expected, except for the knee at Elevation 1095, but low temperatures at the time of construction were responsible for that, as already explained.

That the upper end of Curve No. 6 turns in an up-stream direction, the same as on Curves Nos. 2, 3, and 4, is only logical. All dams—and this is no exception—are provided with an excess of material at the crest, and therefore the stresses and the resulting deformation must be less in this than in lower zones; besides, for Curves Nos. 4 and 6, the outside temperature, and therefore, also, the temperature of the dam body, especially at the crest, was higher than that of the water on May 18th and on June 25th.

This higher temperature of the crest portion tends to accentuate the pointing of the upper portion of Curves Nos. 4 and 6 in an up-stream direction. Curve No. 7 (October 27th, 1915) indicates the effect of lower temperatures of the crest portion, than that of the water and of lower portions of the dam body.

The horizontal scale of these curves is exaggerated 1 200 times, in order to show the results more plainly. In reality, the curves are smooth, although the irregularities, as shown, are present.

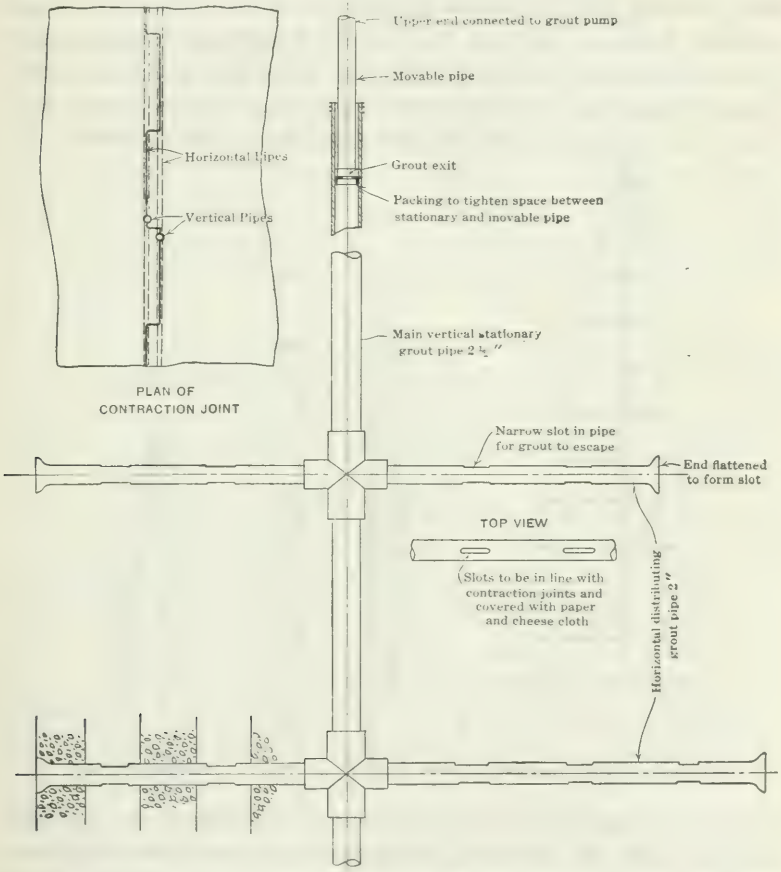
In the foregoing description, some of the actions and behaviors of arch dams have been explained in detail. In the following the method of eliminating some of the undesirable features and making the structure act more like a theoretical arch, than otherwise possible, will be outlined.

By a simple system of iron pipes (shown in Figs. 5 to 8), distributed on the face of the contraction joints, and provided with slots at certain intervals, it is possible to deliver cement grout under pressure into the space (from $\frac{1}{4}$ to $\frac{3}{8}$ in. wide) between adjacent walls of the

FIG. 4.—SALMON CREEK DAM, NEAR JUNEAU, ALASKA. 168 FEET HIGH.

contraction joints, and put initial axial compression into the whole structure, thereby making it act like a solid arch.

At the same time, this grouting will greatly improve the water-tightness of the dam.


Upper end connected to grout pump
Movable pipe
Horizontal Pipes
Grout exit
Vertical Pipes
Packing to tighten space between stationary and movable pipe
Main vertical stationary grout pipe 2 ½ "
PLAN OF CONTRACTION JOINT
Narrow slot in pipe for grout to escape
End flattened to form slot
TOP VIEW
Slots to be in line with contraction joints and covered with paper and cheese cloth
Horizontal distributing 2" grout pipe

ARRANGEMENT OF PIPES FOR GROUTING.

FIG. 5.

It is well known that contraction joints, even of the most efficient design, cannot be expected to be entirely water-tight. The leakage through such joints on large structures may easily amount to from 10 000 to 20 000 gal. daily, and has amounted to as much as 450 000 gal. daily.* This can be eliminated by grouting the joints

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1064.

under pressure; still, the main purpose of this grouting is to compel the arch dam to act as an arch from the very moment the water begins to fill the reservoir.

Forcing the grout into the contraction joints, and keeping it there under pressure until it has set, presses the crown of the arch in an up-stream direction, and puts shear on the foundation in the opposite direction to that due to the water load. It seems logical, therefore, to use such a grout pressure as will force the crown up stream half the total amount of the deflection resulting from a full reservoir. In

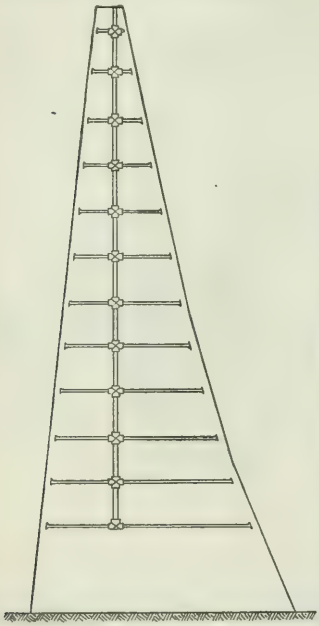


FIG. 6.

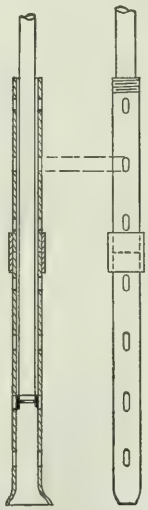


FIG. 7.

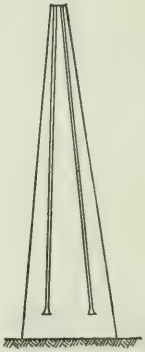


FIG. 8.

this way the maximum shearing stresses are reduced by one-half (reservoir full), the maximum cantilever stresses are reduced by one-half, and, when fully loaded, the arch compression should then be as nearly as possible in accordance with the values obtained from the simple formula:

$$\text{Unit compression} = \frac{\text{Unit water pressure} \times \text{up-stream radius}}{\text{Thickness of section}}$$

A check on the necessary grout pressure can be obtained by sighting across the crown of the arch with a transit, after first establishing the necessary bench-marks.

Great refinement in choice of final grout pressure, however, cannot be counted on, and is not essential. Considering the great influence of the temperature on the total deflection of the arch structure, as can be seen from the curves, especially Curves Nos. 6 and 7, it is necessary to use some judgment as to what the grout pressure should be in each case.

As a general rule, it can be stated that the grouting ought to be accomplished at a time when the structure is cold, and when the joints, therefore, have opened to their maximum extent, permitting easy access for the flow to all parts of thin grout (1 cement to 7 water to start with and thicker to finish). A large dam will generally have the least volume during March, and, therefore, the contraction joints will have opened up a maximum during this time, if the reservoir is empty.

When applying the grout pressure at such a time, it should be kept in mind that the dam body is larger at any other time, due to an increase in temperature, and that, with the cracks closed, the compression per square inch of contact surface will be increased as the temperature of the dam body increases.

Thus far only arch dams have been mentioned, but it would not require much explanation to show the advantage of tight grouted contraction joints on straight gravity dams also.

Grouting the contraction joints under pressure on straight dams throws longitudinal compression on the structure, and enables it to act as a beam held at both ends, at least until the initial compression has been overcome by the beam tension. A trial calculation will prove that the factor of safety has been greatly improved, especially at the lower levels, unless the dam is very long.

The grouting of the joints naturally improves the water-tightness of the dam, and, for this benefit alone, it is worth its cost. Judging from the results of analysis of leakage water through concrete dams, there are good reasons for assuming that a tight dam is going to remain safe for a longer time than a leaky one, although the writer is not attempting to predict the probable natural life of any dam.

A straight dam which has its contraction joints grouted under pressure during the cold season, will automatically have this pressure increased during the warm season, at the rate of about 11 lb. per sq. in. for each degree of rise of temperature of the concrete of 1°

Fahr., as this material tries to expand and cannot. This, of course, should be allowed for, when selecting the grouting pressure for a straight dam. There could be no objection, however, to having a longitudinal compression of, say, from 200 to 300 lb. per sq. in., or even more, on the concrete during the warm season.

A typical grout piping arrangement for a rather large dam section is shown by Figs. 5 and 6.

On the plan of the contraction joint on Fig. 5 there are two vertical feed pipes which are desirable on a joint of the kind shown. Each vertical pipe feeds a system of horizontal pipes arranged at different levels. These horizontal pipes are provided with slots wherever they cross the open space of a joint (Fig. 5). When the pipes are put in place, these slots are covered with paper and cheesecloth to prevent the concrete from entering the pipes from the outside and blocking them. Later, when the contraction joints open, this covering will either adhere to the concrete, or the grout pressure will burst it open.

The pipes cannot be recovered, but will remain forever buried in the dam. They are necessary, however, in order to keep the grout in a liquid condition from the time it leaves the pump until it passes through a slot very close to the desired point of deposit. The iron walls of the pipes do not absorb any of the water in the grout; the absorption, however, commences as soon as the grout leaves the exit slot in the pipe and comes in contact with the concrete walls of the joint. Therefore, this outside grout sets first and shrinks, to a large extent. The grout in the pipe system, being fluid for a long time, is able to follow up the shrinkage and fill out the cavities and keep the joint under pressure until the filler has solidified, in from $\frac{1}{2}$ to 1 hour or more.

To avoid air pockets as much as possible, and otherwise to insure uniform work, the grouting should proceed from the bottom up; therefore a second, and movable, pipe is inserted in the stationary vertical pipe at the time grouting is to be done, as indicated in Fig. 5.

When starting to grout, one end of this movable pipe is to be at the bottom of the vertical pipe, the upper end being connected to the grout pump by a flexible connection. Grout is forced through the movable pipe and flows through the lower Tee, the lower horizontal pipes, and the slots in them, out into the space between the walls of the contraction joints, and rises vertically.

Before grouting commences, the down-stream end of the contraction joints should be caulked (with lead wool) except for small stretches left open for exploration purposes. These are closed when the grout starts to flow out of them. The up-stream end of the contraction joints are generally provided with copper plates or other kinds of effective stops.

When the grout has reached the second story of horizontal pipes, or nearly to it, the movable pipe is raised so that the lower end is just above the Tee connection leading to those pipes. Grout is then forced into them, and, proceeding in the same manner as previously described, the crest is eventually reached. Then, finally, the pressure is kept on until the filler has thoroughly set. It is the grout pressure used on the last, or preferably on the last two joints, spaced one-third of the arch length apart, that determines the initial arch compression, and therefore on these the greatest care should be exercised.

If a dam is so large and high that it takes two or three seasons to build it, it is advisable to grout the lower third of the height at the end of the cold season, before the construction of the dam is continued; otherwise, the lateral expansion (Poisson's ratio), due to the weight of material on top, will tend to close the contraction joints. These spaces will always be narrower close to the foundation than higher up, due to shearing stresses, developed by shrinkage in the concrete close to the rock bottom, tending to prevent complete contraction.

When grouted during cold weather, an arch dam will always be under the influence of compressive stresses, and any knee in the deflection curve, due to inequalities in temperature during the construction period, will be flattened out. All told, the arch dam will act more nearly as intended, carry the load as calculated, and be more water-tight than without the grouting of the contraction joints. A straight dam will have its factor of safety increased, due to the positive beam action, and also be more water-tight.

Figs. 7 and 8 illustrate a simple arrangement of grout piping for the contraction joints of a small dam.

Fig. 9 shows what is regarded as the best arrangement of grout piping for the contraction joints of a dam of medium size (the Carroll Arch Dam). The location of these joints is indicated (not in detail, however) on this figure, which also shows the complete arch, together

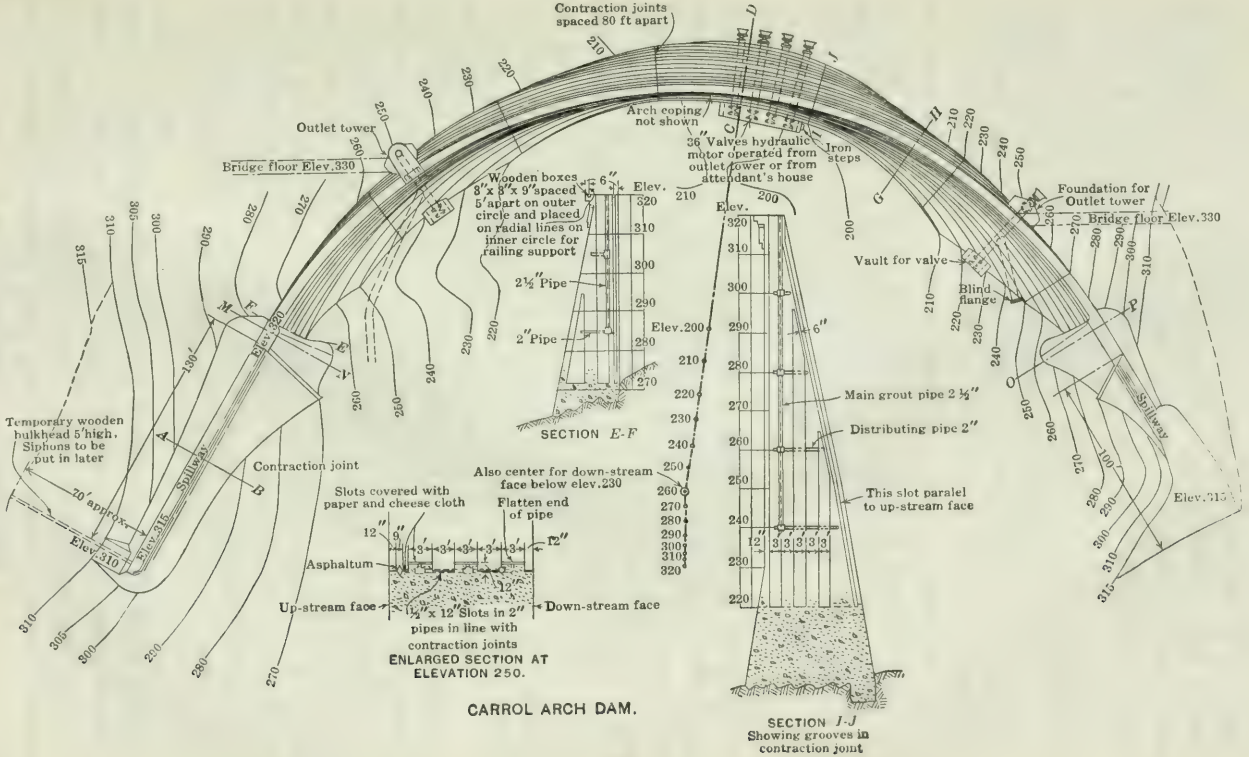

Contraction joints spaced 80 ft apart
Arch coping not shown
36" Valves hydraulic motor operated from outlet tower or from attendant's house
Iron steps
Outlet tower
Bridge floor Elev. 330
Foundation for Outlet tower
Vault for valve
Blind flange
Wooden boxes 3'x 3"x 9' spaced 5' apart on outer circle and placed on radial lines on inner circle for railing support
2½" Pipe
2" Pipe
SECTION E-F
Main grout pipe 2½"
Distributing pipe 2"
This slot paralel to up-stream face
Temporary wooden bulkhead 5' high. Siphons to be put in later
Contraction joint
Also center for down-stream face below elev. 230
Slots covered with paper and cheese cloth
Flatten end of pipe
Asphaltum
Up-stream face
Down-stream face
½" x 12" Slots in 2" pipes in line with contraction joints
ENLARGED SECTION AT ELEVATION 250.
CARROL ARCH DAM.
SECTION I-J
Showing grooves in contraction joint
Spillway
Elev. 315
Elev. 310

FIG. 9.

with its tangent spillways. The enlarged section at Elevation 250 shows the details of a typical contraction joint. Only one vertical pipe is provided, and from this several stories of horizontal pipes reach all parts of the face of the joints.

The longitudinal faces of the individual recesses, making up the contraction joint, are to be made slightly tapering, so as to make sliding more certain. It should be kept in mind, however, that parallel faces will produce the greatest water-tightness, and, therefore, the tapering should be very slight, only enough to take care of construction irregularities.

The deflection curves of Figs. 1 and 2 were made under the direction of Mr. H. L. Wellenberg, formerly Chief Engineer of the Alaska Gastinau Mining Company, Juneau, Alaska. Measurements on other dams have been made, but those given are characteristic of the deflection of the crown of an arch dam, due to changes in load and temperature.

DISCUSSION

Mr. Creager.

W. P. CREAGER,* M. AM. SOC. C. E. (by letter).—It is very interesting to note that, of the hundreds of arch dams which have been constructed, no record of a single failure can be found. This is particularly remarkable because of the fact that less is known regarding the distribution and intensity of stresses in arch dams than in any other type of masonry dam. Perhaps this signal success with such dams can be accounted for by the remarkably large factor of safety usually adopted in the designs.

With complete information as to the distribution of stresses, as effected by a combination of water load, base shear, and expansion and contraction, the economic value of this already useful type of dam would be augmented considerably.

Although engineers may never hope to obtain perfect knowledge of the subject, the results of deflection measurements, so well described by Mr. Jorgensen, assist to a marked degree and make for economies in future designs. It is to be hoped that more data of this kind will be forthcoming.

Mr. Jorgensen's ideas regarding the desirability of initial up-stream deflection, before filling the pond, is based on sound reasoning. The greatest uncertainty regarding the distribution of stresses in arch dams is the result of the unknown effect of the so-called "cantilevering action", or horizontal shear at the base, which is caused by a down-stream deflection of the upper part of the dam relative to that of the lower part, held rigid by the foundation. An initial deflection in an up-stream direction causes a negative shear which is eliminated on loading the dam, resulting in a negligible or at least greatly reduced positive shear and a better distribution of arch stresses.

If, as is customary, the dam is constructed during the warm season, the deflection during the cold season, after loading, will be considerably more than that theoretically due to the water load alone, on account of the shrinkage of the concrete in setting, and that due to a reduction in atmospheric temperature. The writer has been considering for some time the advisability of providing contraction joints having clear openings of a foot or more. If, before the pond rises, these contraction joints are filled with concrete during the cold season, the deflection due to the setting of the concrete and a reduction in atmospheric temperature would be obviated with beneficial results.

The advantage of carrying this remedial precaution to the extent suggested by Mr. Jorgensen depends greatly on the cost and effectiveness of such work. It would be interesting to know the results of Mr. Jorgensen's experience with the method of grouting proposed, par-

* New York City.

ticularly as regards the leakage of grout from the joint. Such leakage, if excessive, would be not only costly, but would greatly disfigure the down-stream surface of the dam. Mr. Creager.

S. H. Woodard,* M. Am. Soc. C. E. (by letter).—The author has developed a serious question to be considered in designing and building arch dams. Mr. Woodard.

The general knowledge of the fact that arch dams must shorten in length by shrinkage, both when the concrete is setting and during cold weather, and that this shortening must cause re-distribution of stresses, has been in the mind of every thoughtful designer. It has been observed that nearly all straight dams and all arch dams have developed vertical cracks in cold weather, but, so far as the writer is aware, the record of actual observed deflections of an arch dam has not been published before.

The data in the paper are very useful, and probably all will agree that some steps may well be taken to accomplish what is hoped for from the grouting scheme proposed. The writer is of the opinion that, if any perfectly reliable method of accomplishing, or nearly accomplishing, this can be devised, it may well be used in preference to the grouting scheme. It is evident that, if the construction joint is only partly filled with grout, leaving certain areas without bearings when the joint is closed under pressure, the dam will be worse off than before grouting. Unless extraordinary care is taken, it will never be known whether or not the joint has been filled uniformly with grout.

The writer believes that, in most cases, wherever feasible, it will be better practice to leave narrow gaps at, say, 50-ft. intervals, and close them with concrete placed in the early spring after the dam has stood through the winter.

This, of course, will not introduce the initial stress which is hoped for from the grouting method, but here, again, unless the initial stress is uniform, the dam is better off without than with it, and there would always be some doubt on that point.

W. H. R. Nimmo,† Assoc. M. Am. Soc. C. E. (by letter).—The deflection curves presented by the author form a valuable contribution to the available data on arch dams, and it is to be hoped that other engineers having charge of large arch dams will make similar observations. If an analysis is made of the arch and cantilever stresses at different levels in any arched dam, the results will indicate a deflection curve of the same general shape as that of the curves obtained by the author, this being especially true of the movements of the crest. The peculiar bend in the curves at Elevation 1095 is significant, as it indicates the extent to which conditions during construction may influence the subsequent behavior of the dam. Mr. Nimmo.

* New York City.

† Hobart, Tasmania.

Mr. Nimmo.

In his discussion of the late Mr. L. A. B. Wade's paper, "Concrete and Masonry Dam-Construction in New South Wales",* Mr. De Burgh describes a concrete arch dam, 38 ft. high, built for domestic supply at Barren Jack, and the deflection curves shown in Fig. 10 are reproduced from his discussion. At the right abutment, the stress is distributed into the rock by a concrete abutment and wing-wall, but it

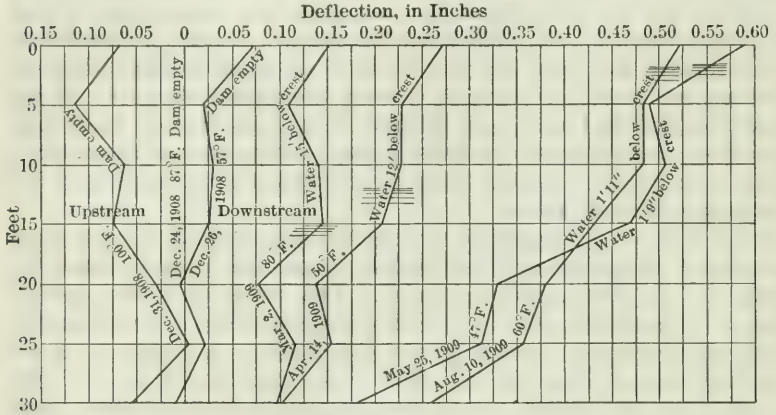


Fig. 10.

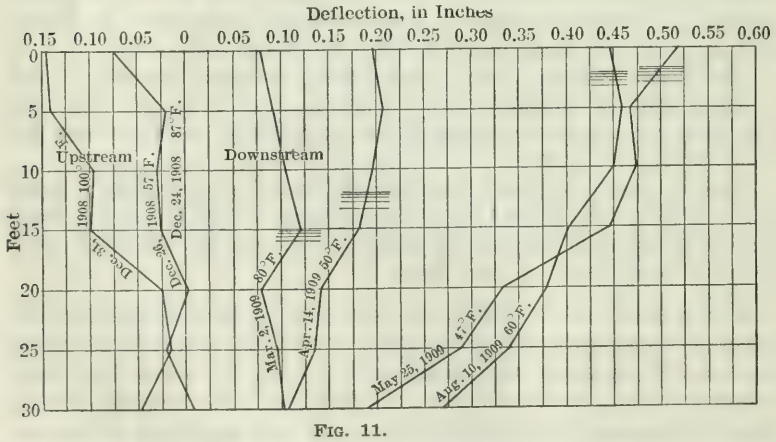


Fig. 11.

is stated that there is no indication of any yield having taken place at this point. The movements of the crest, however, are not such as theory would lead one to expect. The dam was completed in December, 1908, and the first set of measurements, used as a datum in Fig. 10, were taken on December 24th, 1908. If these measurements were taken very soon after placing the last lift of concrete, it is probable

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII.

that the initial shrinkage of the concrete would unduly influence the movement of the crest relative to the rest of the dam. In Fig. 11 the writer has replotted the curves, using the measurements of December 26th, 1908, as a datum, and it will be seen that the movements of the crest become more logical. The two left-hand curves of Fig. 11 show the influence of a decided rise in temperature on the thin crest (in this case 2 ft. thick). In the two right-hand curves of Fig. 11, the water is practically at the same level in each case, and, although the temperature of the air on May 25th, 1909, was below that of August 10th, 1909, the actual temperature of the water and of the thicker concrete in the lower levels would probably be higher in May than in August. The thin crest, however, would respond more readily to a fall in temperature, and this appears to be the reason for the two curves crossing each other. It is of interest to notice that the curves at a point 20 ft. below the crest show a bend somewhat similar to that shown by the author's curves at Elevation 1095, but no information is given as to temperatures during construction. This dam contained a considerable percentage of longitudinal and vertical reinforcement in the form of old rails, but its presence does not appear to have had any effect on the general shape of the deflection curves. Mr. Nimmo.

L. R. Jorgensen,* M. Am. Soc. C. E. (by letter).—Both Mr. Creager and Mr. Woodard have made the suggestion that narrow gaps, say, 12 in. wide, should be left for contraction joints which should be filled during the cold season, before the pond rises. Mr. Jorgensen.

It is true that such a wide crack could be filled more uniformly over 100% of the bearing than one only ¼ to ⅜ in. wide, but it would require twice the quantity of formwork to make this wide crack than to make the ordinary contraction joint, and the cost of this extra formwork would be considerably more than that of iron pipes for grouting purposes.

The grouting process does not take long to complete, and the richer grout will attain a high compressive strength in a short time, making it possible for the structure to withstand full water pressure very soon after grouting.

Mr. Woodard points out that, unless extraordinary care is taken, it will never be known whether or not the joint has been filled uniformly with grout. This, of course, is true.

The idea of the grout-pipe system has been developed largely from observations of how not to do grouting. The iron pipes are absolutely necessary, in order to be reasonably sure that too much area is not left without bearing.

One cannot be sure that the bearing of the contraction joint is 100%, but a water-tight joint can be obtained with less, as the voids would not necessarily connect with one another.

* San Francisco, Cal.

Mr. Jorgensen.

Arch stresses are generally kept low enough so that an accidental higher bearing stress can be taken care of with perfect safety. The material composing the ¼-in. inserted "key-stone" is well supported, and, therefore, can carry safely a higher stress than ordinary arch compression.

Mr. Nimmo's curves, shown in Fig. 11, correspond fairly well with what one would expect under the varying conditions of temperature and load.

The deflection of the crown of the Lake Spaulding Dam has been measured, from time to time, at a point 32 ft. down from the crest of the present dam (260 ft. above river level and 280 ft. above bed-rock), and at the crest.

The deformation of the concrete at the crest is more subject to variation of the temperature of the air, than at the point 32 ft. down, where the arch is thicker, which is only logical.

After the Lake Spaulding Dam was built to a height of 225 ft. above river level, in 1913, at a rather high rate of speed, the zero point for the deflection at the crest was set on November 25th, with the reservoir practically empty, but with all the chemical heat still in the concrete.

Three months later, the reservoir was full, and the cold water and air (February) had absorbed all or most of the chemical heat. The total deformation, due to the water load, the disappearance of the chemical heat, and the natural shrinkage, was 3.44 in.

The chemical heat had evidently caused the crown to be pushed in an up-stream direction about 2½ in., as the maximum deflection since has been less than 1 in. In 1916, 35 ft. were added to the top, and, at present, 15 ft. additional are being constructed.

During the last two winters, the contraction joints have been grouted with the exception of two. This grouting has had the effect of diminishing the deflection, which it should, and on July 8th, 1919, with the reservoir practically full, the point 32 ft. below the crest deflected only $\frac{3}{16}$ in. and the one at the crest only $\frac{1}{16}$ in., this latter point being more subject to the higher temperature in July.

The water load generally comes on in the spring, at the same time as the increase in temperature. When the two remaining ungrouted contraction joints are filled, it can be expected that, under the conditions stated previously, there will be no deflection, in which case the arch would act as a theoretical arch, with no secondary or cantilever stresses.

Should the reservoir happen to be full in the winter, there would still be some deflection, as then the deformation due to load would not be compensated by expansion due to increase in temperature; in fact, thermostats show a yearly variation of 20° Fahr. inside the dam, 24 ft. down from the 260-ft. crest, but the deflection would be less than if no grouting of the contraction joints had been done.

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Paper No. 1432

SEWAGE AND WASTES DISPOSAL FOR THE UNITED STATES ARMY*

By Leonard S. Doten,† M. Am. Soc. C. E.

With Discussion by Messrs. Leonard Metcalf, Nicholas S. Hill, Jr., Almon L. Fales, Kenneth Allen, G. Bertram Kershaw, Edmund B. Besselievre, George T. Hammond, A. L. Sherman, J. F. Brown, Lynn E. Perry, and Leonard S. Doten.

Synopsis.

This paper presents a review of the work of the Sanitation Section of the Construction Division of the United States Army. The engineering problems are briefly discussed, and the designs of plants and equipment for the treatment of wastes and sewage in camps and cantonments of the United States Army are described. A comparison is made between the characteristics of municipal sewage and camp sewage, data being presented which show the exceptionally high concentration of camp sewage. Reference is made to special studies that were required to devise means to treat these wastes adequately.

The departure from previously recognized standard practice, in some instances, in the design of sewage treatment plants, and the reasons for it, are explained, and the results obtained at various camps are given.

Attention is called to the complexity of the sewage disposal problem, the field for careful research work in perfecting the processes

* Presented at the meeting of December 18th, 1918.

† Major, Q. M. C.; Officer in Charge of Sanitation Section, Engineering Branch, Construction Division, U. S. Army.

in use, and the necessity for the development of satisfactory plants at lower construction cost.

Although the Construction Division (Cantonment Division) of the Army was organized only a very few weeks in advance of the beginning of construction work on the great military camps and cantonments in 1917, a great deal of study, nevertheless, was given to all problems dealing with sanitation. It was recognized from the first that the correct solution of the sanitary problems was extremely important. Many references were made to the unfortunate conditions as to sanitation which obtained during the Spanish-American War. The experience gained by the War Department in establishing and maintaining camps along the Mexican Border and in Mexico was of great value in the preparation of plans for the large camps constructed during 1917. As a result of this experience, each cantonment and officers' training camp was provided with a complete system of sewerage, including modern plumbing equipment. Authorization of complete water-carriage sewerage systems for the tent camps originally constructed for the National Guard was not secured until the beginning of the season of 1918. The systems are now complete.

In sanitation, as well as in problems of structural and other branches of engineering, the engineers of the Construction Division were required to devise plans and prepare general specifications which would meet all reasonable requirements and at the same time facilitate the execution of the work as much as possible, as speed in construction was of the utmost importance.

In the preliminary work it was soon found that it would be necessary to depart quite radically from many of the recognized standards of engineering practice. This will be shown later by illustrations of designs of structures and references to specifications.

The work with which the writer has been most intimately connected consisted in the preparation of standard designs and general specifications of water-carriage sewerage systems, sewage treatment plants, incinerators, transfer stations for camp wastes, drainage and anti-mosquito work, and the design of devices for reclaiming waste products having recoverable values.

In June and July, 1917, much study was given to the problem of the collection and disposal of garbage and other wastes at the sixteen

National Army Cantonments. As a result of this, it was decided to classify the wastes and separate the waste materials in such a manner as to secure the best possible returns from their sale. This was an important conservation measure. Favorable consideration was at first given to the establishment of reduction plants for the disposal of garbage. Later, however, it was found impracticable to adopt this method of disposal, because of the abnormally high cost of equipment and the long time for delivery, resulting from war conditions.

The scheme finally adopted consisted in collecting the garbage by the duplicate can system, collecting and transporting all valuable waste materials to a central transfer station, where they were delivered to the contractor, and he was required to haul all organic waste materials to a point sufficiently beyond the limits of the Government reservation to avoid any possibility of a nuisance being created, and to protect the health of the troops. Incinerators were built at each of the transfer stations, to be used in the destruction of such waste materials as had no commercial value, and also to be used in the case of an emergency when, for any reason, the contractor failed to remove the garbage promptly.

In some instances the contractors disposed of the garbage by means of reduction plants in large near-by municipalities; and in other cases this material was fed to hogs.

The waste materials were classified as follows: Garbage, paper, bottles, tin cans, bones, fats and grease, and manure.

The sale of these materials, rather than their destruction by incineration, has resulted in a large revenue to the Government, instead of a large outlay for equipment and fuel, which would have been required for disposal by incineration. For July, August, September, and October, the sales of garbage and other wastes included in the preceding classification amounted to $360 570. At this rate, the total revenue to the Government from this source for the year would be about $1 082 000.

The conditions at military camps are especially favorable for the use of the duplicate can system of collection, as there are so many units of approximately equal population. From the sanitary standpoint this system also possesses a considerable advantage over the usual method of collection, due to the fact that the filled cans are transported directly from company kitchens to the transfer station, thus eliminating

the possibility of strewing waste materials on the grounds in the vicinity of camp buildings. The cans are thoroughly cleansed at the transfer station and are returned to the company kitchens at the time the filled cans are being collected.

The transfer stations (Plate III) are designed so as to provide the best possible facilities for the rapid transfer of wastes from Government vehicles to contractors' vehicles. Rooms are provided for the storage of non-perishable materials, such as paper, bottles, and tin cans, and a washing room is also provided for the cleansing of cans. The water for washing the cans is heated with pipe coils placed in an incinerator which is an adjunct to the transfer station.

Special designs for can-washing equipment and incinerators have been developed by the Construction Division since some of the original plants were built. The cans are placed in long vats containing hot water, in which special cleansing compounds have been dissolved, and allowed to remain a sufficient time to loosen the organic and greasy material from the surfaces. They are then taken out of the vat and placed in a small chamber, constructed of light-weight steel plates, and subjected to a strong spray of hot water, both internally and externally, which thoroughly cleanses them. The waste products resulting from this process pass through a large grease-intercepting tank before being discharged into the sewerage system.

The can-washing equipment at the transfer stations (Fig. 1), as constructed in 1917, consisted of large circular tanks for hot water only. Later, at a few plants, mechanically-operated washing equipment was provided, but more recently the type of apparatus previously described has been used as a standard at all transfer stations.

During the season of 1918, special designs were prepared for incinerators which would meet all requirements for the incineration of both animal and vegetable wastes having no commercial value.

In determining the capacity of an incinerator for a camp, it is assumed that the quantity of material to be handled would be 1 ton per 1 000 men per day, and that 8 hours would be the operating period per day.

The designs provide for ample firing and combustion chambers; a grate surface for natural draft to burn 35 lb. per sq. ft. per hour, and for forced draft, 60 lb. per sq. ft. per hour; a drying hearth to facilitate the cremation and to prevent liquids from reaching the

PLATE III.
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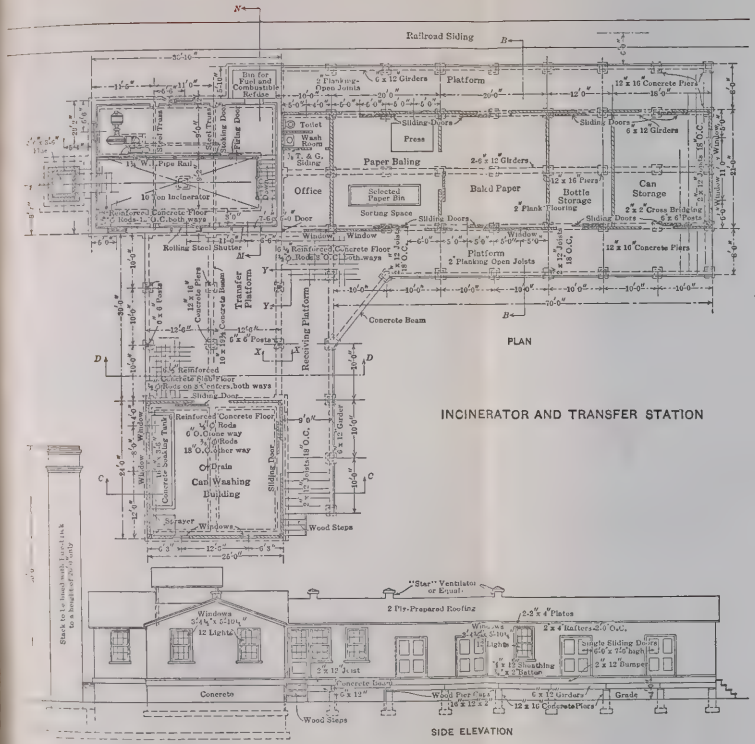
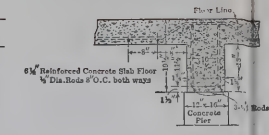
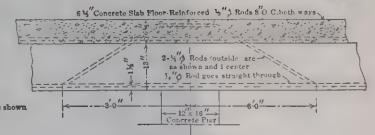
DOTEN ON
SEWAGE AND WASTES DISPOSAL.

INCINERATOR AND TRANSFER STATION

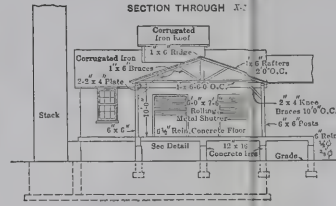




SECTION THROUGH X-X


6¾" Concrete Slab Floor Reinforced ⅝" Rods 8'-0" C both ways
Rods outside are as shown and 1 center
Rod goes straight through
Concrete Pier

SECTION THROUGH Y-Y



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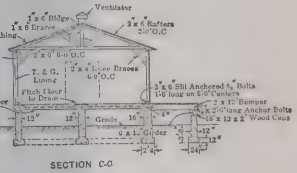

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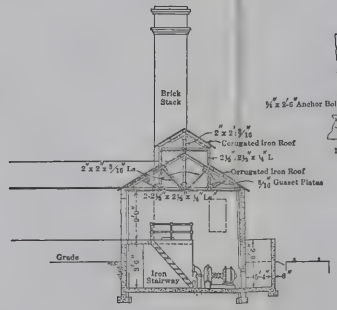

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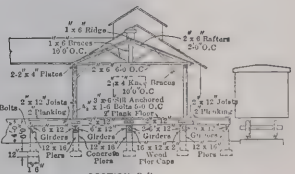


SECTION B-B

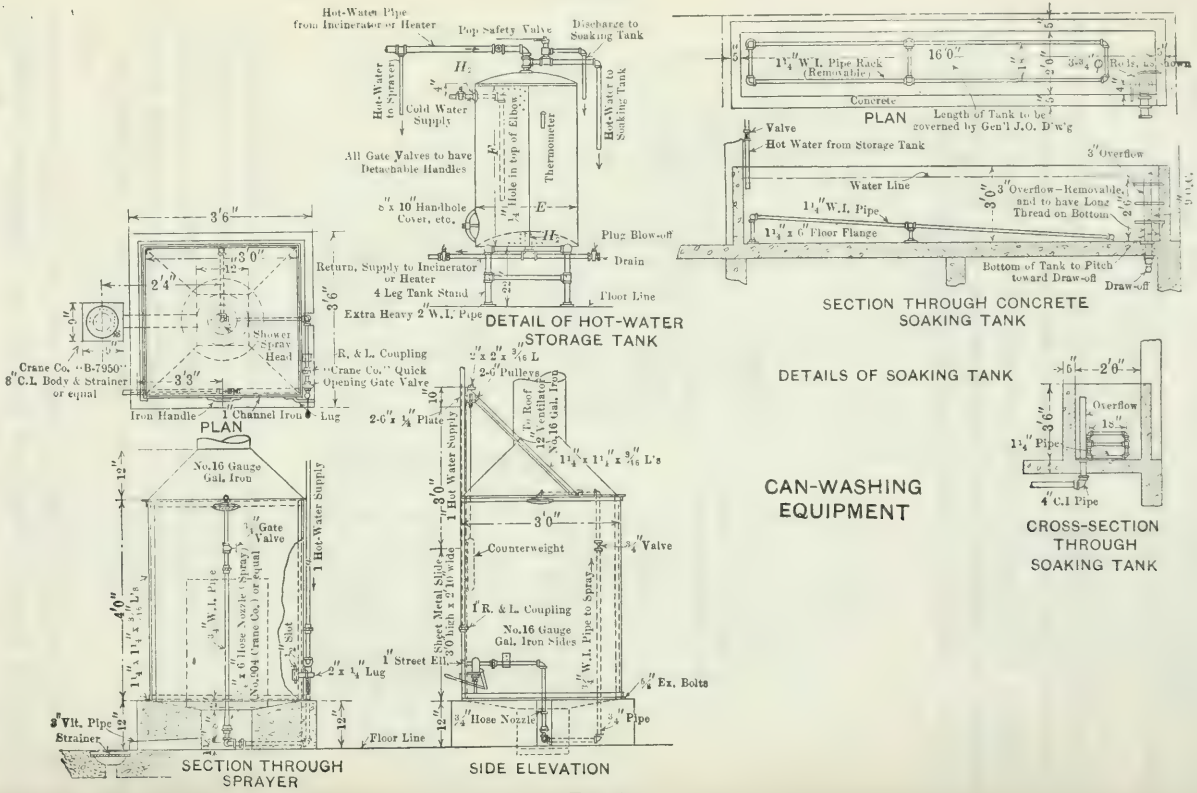

Hot-Water Pipe from Incinerator or Heater
Pop Safety Valve
Discharge to Soaking Tank
Hot-Water to Sprayers
Cold Water Supply
Hot-Water to Soaking Tank
All Gate Valves to have Detachable Handles
Thermometer
8" x 10" Handhole Cover, etc.
Plug Blow-off
Drain
Return, Supply to Incinerator or Heater
4 Leg Tank Stand
Floor Line
Extra Heavy 2" W.I. Pipe
DETAIL OF HOT-WATER STORAGE TANK
3'6"
3'0"
2'4"
Shower Spray Head
Crane Co. "B-7950" 8" C.I. Body & Strainer or equal
3'3"
R. & L. Coupling
Crane Co. "Quick Opening Gate Valve
Iron Handle
1 Channel Iron Lug
PLAN
2"x 2"x 3/16" L
2-6" Pulleys
2-6"x 1/4" Plate
No. 16 Gal. Iron
1 1/4"x 1 1/4"x 3/16" L's
No. 16 Gauge Gal. Iron
1" Hot Water Supply
3/4" Gate Valve
3'0"
Counterweight
3/4" Valve
1" W.I. Pipe
1" x 6" Hose Nozzle (Spray) (No. 804 Crane Co.) or equal
Shut
1" R. & L. Coupling
No. 16 Gauge Gal. Iron Sides
1" Street Ell
2"x 1/4" Lug
3/4" Ex. Bolts
2" Vlt. Pipe Strainer
3/4" Hose Nozzle
3/4" Pipe
Floor Line
SECTION THROUGH SPRAYER
SIDE ELEVATION
1 1/4" W.I. Pipe Rack (Removable)
16'0"
3 3/4" Rods, Lg. 5'0" Down
Concrete
PLAN
Length of Tank to be governed by Gen'l J.O. D'w'g
Valve
Hot Water from Storage Tank
3" Overflow
Water Line
3'0"
3" Overflow—Removable, and to have Long Thread on Bottom
1 1/4" W.I. Pipe
1 1/4" x 6" Floor Flange
Bottom of Tank to Pitch toward Draw-off
Draw-off
SECTION THROUGH CONCRETE SOAKING TANK
DETAILS OF SOAKING TANK
6"
2'0"
Overflow
18"
3'6"
1 1/4" Pipe
4" C.I. Pipe
CROSS-SECTION THROUGH SOAKING TANK
CAN-WASHING EQUIPMENT

FIG. 1.

grates; dampers to control the draft; and ample stack area and height to prevent back-firing and produce an everage velocity of gases of approximately 20 ft. per sec. Plate IV shows a 20-ton incinerator plant.

Although provisions were made for intercepting the grease from kitchen waste-water at the time the original cantonments were constructed, it was found, later, that the commercial type of trap which had been used was not effective, owing partly to faulty design and partly to limited capacity, and, early in 1918, it became evident that a more

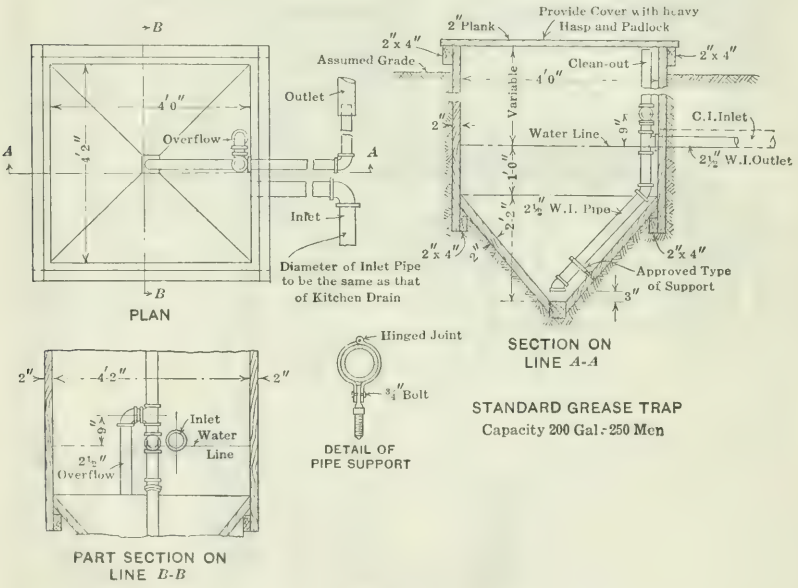
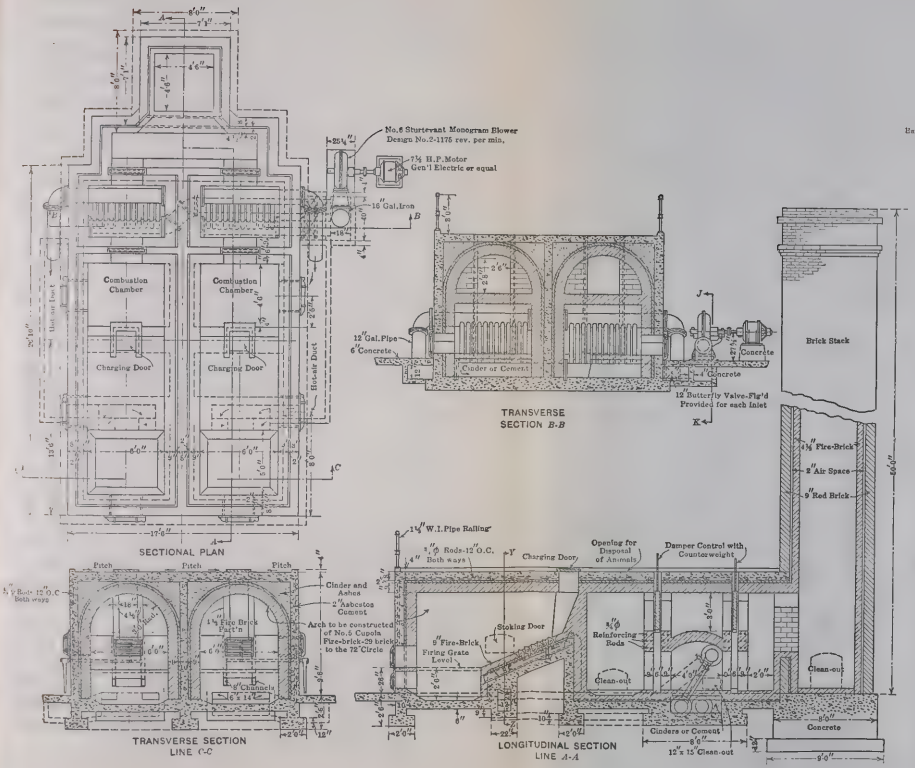


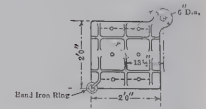
FIG. 2.

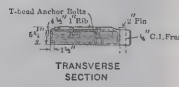
effective type should be used, in order to prevent the sewer pipe lines from being filled with grease deposits, and to relieve the excessive grease load on the sewage treatment plants. It was also recognized that the recovery of such large quantities of grease would result in a considerable revenue to the Government and would be in the interest of conservation.

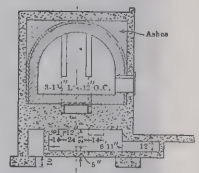
Traps having a holding capacity of approximately $\frac{2}{3}$ gal. per capita were designed, and have been placed in nearly all the cantonments. (Fig. 2.) The special features of these traps are: the relatively large

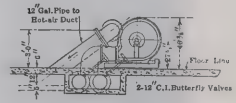
PLATE IV.
TRANS. AM. SOC. CIV. ENGRS
VOL. LXXXIII, No. 1432.
DOTEN ON
SEWAGE AND WASTES DISPOSAL.



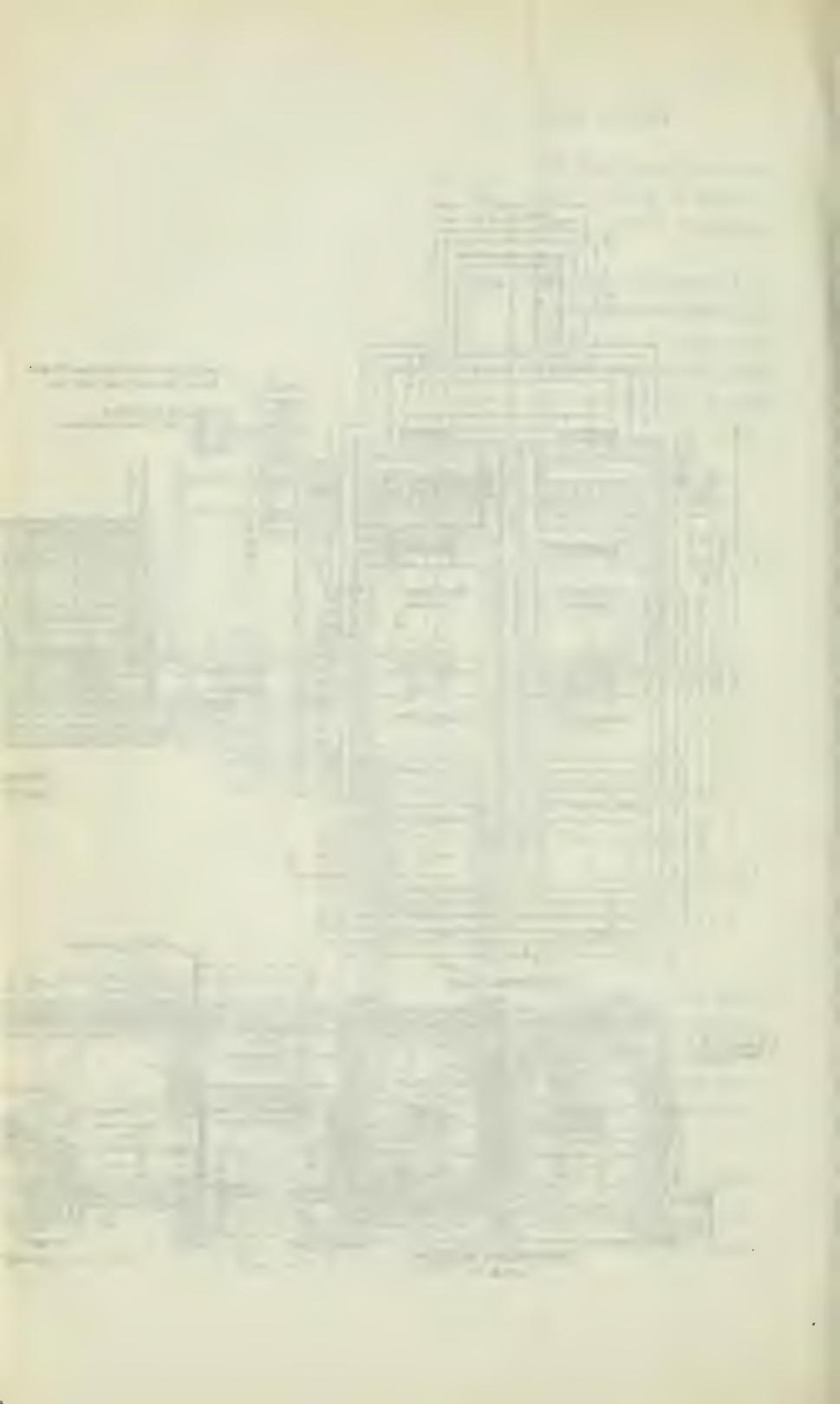








20-TON INCINERATOR PLANT



area for the collection of grease; the steep-sided pyramidal, or conical, bottoms; and the small diameter of the eduction pipes which remove the settleable solids. The efficiency of these traps has been determined by several series of carefully made tests which show conclusively that the average efficiency under the usual operating conditions is at least 95%, and, in some instances, 99% has been recorded.

Grease recoveries by these traps range from 10 to 25 lb. per capita per year. The average is estimated to be between 12 and 14 lb. Thus, it will be seen that the revenue obtained from this waste material alone is very large at present, owing to the high prices now being paid for this product. Rendering plants have been built at some of the camps for purifying the grease obtained from the traps, in order to secure a very much higher price for the product than would be obtainable if marketed in the crude state.

Numerous determinations of the grease content of the sewage have been made at the camps, and indicate that 150 parts per million is a fair average quantity. It will be noted that this is very greatly in excess of the quantities usually found in municipal domestic sewage, the average of the latter usually ranging between 20 and 30 parts per million. This is one of the most important characteristics of military camp sewage. The following are fair averages of some of the other constituents of camp sewage:

Total organic solids.....	700	parts	per	million
Total suspended organic solids.....	464	"	"	"
Free ammonia.....	48	"	"	"
Total organic nitrogen.....	32	"	"	"

It will be seen, therefore, that the sewage is highly concentrated, and that the quantity of suspended organic matter is exceptionally high. The average quantity for nine American cities is 125 parts per million; for three English cities, 237; and for all cities, both foreign and American, it is 190. The conclusion, therefore, is that camp sewage contains about 3.7 times the average for nine American cities, 2.4 times the average for all cities, and nearly twice the average for English cities. The strength of the sewage is undoubtedly largely due to the relatively small per capita water consumption, certain waste products from kitchens, such as garbage and grease, and to the fact that the population is made up wholly of adults.

The sewage flow per capita per day ranges from approximately 35 to 60 gal. The fluctuations in the rate of flow, as shown by the graphs on the chart, Fig. 3, are much greater than in municipal systems.

When the general specifications for sewerage were prepared, it was anticipated that the maximum rate of flow from a divisional cantonment would be between 2½ and 3 times the average rate per 24 hours. Since that time, tests on several of the systems have shown that this estimate closely approximates the actual results. The maximum rate of flow occurs during shower-bath periods, and this continues for a period ranging from 40 to 60 min. at each unit. In the original speci-

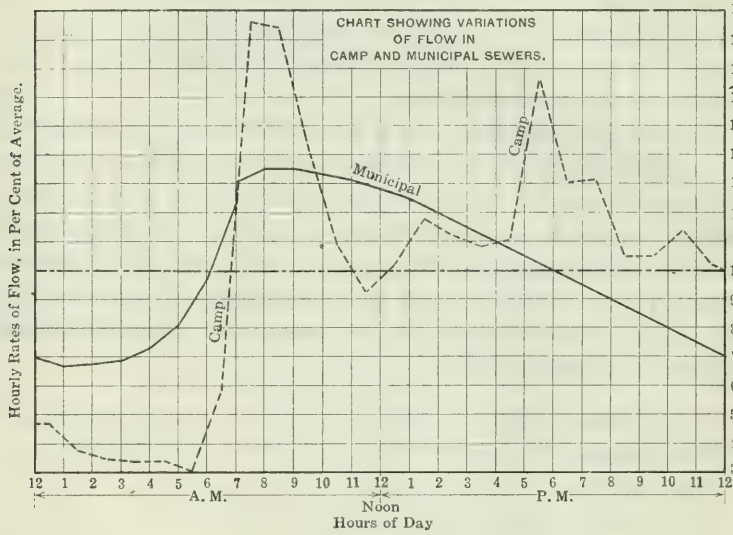


FIG. 3.

fications, certain requirements as to the capacities of the sewers were adopted. The assumption was made that it might be necessary to limit the use of shower baths in a divisional cantonment to one infantry brigade at a time, in order to avoid excessively high rates of flow, and, consequently, the construction of very large trunk and outfall sewers. As the specifications were drawn in advance of the selection of the sites for the cantonments, and, consequently, the exact location of the several units constituting a divisional cantonment had not been determined, it was impossible to calculate by a more exact method the necessary capacities of the sewer lines. Fortunately, however, the adoption of

this method resulted in a very satisfactory adjustment of sewer capacities.

As a result of the data which have since been accumulated, relative to the rates of flow in the several parts of cantonment systems, from tests made in the original sixteen cantonments, it has been possible during 1918 to check this method by a somewhat more scientific one involving as factors the maximum rates of discharge from individual units, the maximum length of time such discharges continue, and the distance that sewage would flow in that interval of time. This method of computing the sizes of sewers, in cantonments having approximately the straight-line type of layout, gives results which are materially lower than are obtained by using the former method. It is of less importance—although of considerable value—as a check on the original method in layouts approximating the horse-shoe type. In such cases, the concentration of flow at certain points is extremely high. In cantonments having a population of approximately 40 000 to 50 000, the maximum diameter for the outfall sewer is usually between 24 and 30 in.

The sewer connections to the lavatory buildings are 6 in. in diameter. In a few instances, the connections for kitchens have been 4 in. in diameter. The specifications adopted by the Construction Division contain a tabulation of minimum gradients, for sewers of different sizes, which will produce self-cleansing velocities. The calculations are based on a mean velocity of approximately $2\frac{1}{4}$ ft. per sec., with the sewer half full or full. All computations for capacities are based on sewers flowing approximately full during the highest rates of discharge.

The specifications require that the average distance between manholes shall be 350 ft. The manholes are of the simplest possible design, and are either square or round, depending on the kind of material used in construction. The sides are vertical, except where it is necessary to use standard cast-iron frames and covers. It has been the practice of the office to avoid, as far as practicable, the use of cast-iron frames and covers, both on account of the high cost of such material and the impracticability of using small covers on shallow manholes. By the adoption of wooden covers it is possible to have vertical walls, and thus improve the facilities for inspection and the introduction of sewer-cleaning apparatus. The wooden covers are very much cheaper than cast iron. A very important consideration, also, is the fact that it is necessary to lock the camp sewer manhole covers, in

order to prevent using them as receptacles for all kinds of refuse, such as old clothing, garbage, etc.

Vitrified clay sewer pipes, with the standard type of Portland cement joints, have been used almost wholly. In a few systems, concrete outfall sewer pipe lines have been constructed.

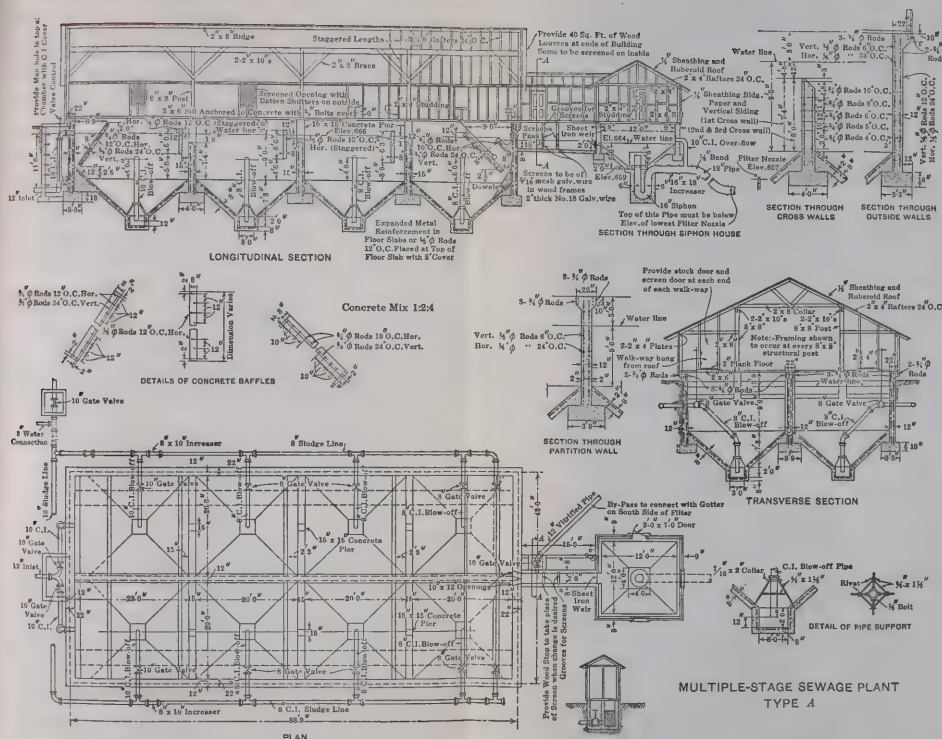
Sewage Treatment Plants.

Excepting in a few instances, where the type of sewage treatment works was pre-determined by soil characteristics or peculiarities of local practice favored by State Boards of Health, the standard type of tanks and filters adopted by the Construction Division has generally been used. Different types of tanks were very carefully considered before standards were adopted, taking into account the peculiar characteristics of the sewage, so far as known at that time, the necessity for rapid construction, and the comparative cost of different designs. The standard tanks are designed on the simplest possible lines, in order to facilitate construction and reduce the cost to a minimum. They are shown on Plate V, and are of a mutliple-compartment horizontal-flow type, provided with baffles and cross-walls, or weirs, in such a way as to facilitate sedimentation and the withdrawal of sludge. The bottoms of compartments are pyramidal.

In operation, the tanks are comparatively easy to control, especially with domestic sewage of average quality. In this respect they have a marked advantage over the older types.

Among other types considered for use at cantonments was the Imhoff tank, but the principal reasons for not adopting it were: (1) the high cost of construction, estimates showing that their average cost would be approximately three times as much as for the single-story tank; (2) the time required for construction would be at least twice as great as for the latter; and, furthermore, it would be less well adapted for treating the fresh uncomminuted camp sewage (Fig. 4). In the light of more recent developments, it is fortunate, for another reason, that the Imhoff tank was not adopted, namely, the difficulty of using a short-period, sedimentation tank which is incapable of protecting filters against excessive quantities of grease carried in the sewage.

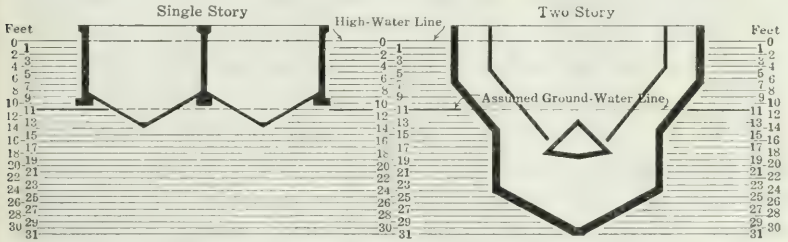
The practice of the Construction Division has been to build treatment works only at camps where it was impracticable to construct an outfall sewer to connect with a city system, or to dispose of the sewage


LONGITUDINAL SECTION
SECTION THROUGH SIPHON HOUSE
SECTION THROUGH CROSS WALLS
SECTION THROUGH OUTSIDE WALLS
DETAILS OF CONCRETE BAFFLES
Concrete Mix 1:2:4
SECTION THROUGH PARTITION WALL
TRANSVERSE SECTION
PLAN
SECTION ON LINE A-A
DETAIL OF PIPE SUPPORT
MULTIPLE-STAGE SEWAGE PLANT
TYPE A

by discharging it into a large body of water, or a river where the dilution factor would be satisfactory.

In most cases, the treatment works consist of sewage tanks and chlorinating equipment only. In comparatively few cases has it been necessary to put in secondary treatment works consisting of trickling filters and secondary sedimentation tanks.

The trickling filter, Fig. 5, owing to its large unit capacity, simplicity of operation, and low cost, was adopted as the best means for secondary treatment. The designs for these filters differ in several important details from the usual municipal engineering practice. Deep, expensive excavations were avoided wherever practicable, advantage being taken of the conformation of the site. This was also true of the retaining walls, which were omitted wherever possible, terminating the filtering material with steep slopes.



CROSS-SECTIONS OF SEWAGE TANKS OF EQUAL CAPACITY SHOWING THE RELATIVELY GREAT DEPTH OF A 2-STORY TYPE AS COMPARED TO THE STANDARD TYPE. THE COST OF THE FORMER TANK IS GREATLY INCREASED ON ACCOUNT OF DEPTH BELOW GROUND-WATER LINE.

FIG. 4.

The filter has an undulating floor, the distributor pipes being placed on the summits and pipe-covered gutters in the depressions. Thus, the expensive system of under-drains, which is usually an important feature in plants for municipalities, was eliminated. The inverts of the drains and gutters are in a plane parallel to the plane passing through the spray nozzles. This plane has an inclination approximately equal to the hydraulic gradient of the distributing system during the average rate of discharge from the nozzles.

The filtering material consists of hard, durable, crushed rock or slag, the pieces of which will pass a screen of 2½-in. openings and be retained on a screen having 1½-in. openings. Relatively coarse rock, having little variation in size of pieces, is favored, as such material has large voids, which are not completely filled with accumulations or

films, and consequently admit of a free circulation of air. The first filters constructed were 6 ft. deep. In a few plants for hospitals, 8-ft. filters were used. The present standard depth for trickling filters is 5 ft.

Dosing or Siphon Chambers.

The writer's experience and observations led him to devote a great deal of attention to the design of siphon chambers, with a view to securing the highest possible efficiency in the operation of trickling filters. A radical change has been made in the dosing period, reducing it from 4 or 5 min. to a maximum of 1½ min. for filters of all depths. Filters not more than 5 ft. in depth, when dosed in this manner, will yield a uniformly stable effluent, which is generally very low in turbidity. The degree of nitrification and oxidation which takes place in such a filter is equal to that obtained from deeper filters, as is illustrated by the following figures taken from a recent report on a series of tests on the Fort Myer, Va., filter. In this report, a comparison is made between the results obtained from the experimental trickling filter at Boston in 1907, and those from the Fort Myer filter. The tank effluent, in so far as the organic nitrogen and free ammonia were factors, was practically the same in the two cases. Expressed in parts per million, the filter effluent from the Boston plant contained organic nitrogen, 3.4; free ammonia, 12.0; nitrites, 0.8; nitrates, 5.3. The Fort Myer filter effluent contained: organic nitrogen, 0.7; free ammonia, 1.8; nitrites, 0.4; nitrates, 25.7. The Fort Myer effluent averaged in dissolved oxygen 42% of saturation. This filter, in the same series of tests, gave an average reduction of 95% in total count of bacteria and 92% reduction in *B. Coli.* Numerous tests, made during the past 8 months, on the effluent from this filter, have shown 100% stability at all times.

During the season of 1917, all sewage treatment plants were designed by the supervising engineers in the field, following the requirements of the general specifications and standards which had been adopted by the Construction Division. During the season of 1918, however, the engineering force of the Sanitation Section of the office has been largely augmented, and all designs for treatment works have been prepared in the Washington office. This has been found to be a more satisfactory arrangement than that of the previous season.

The following assumptions are made in the design of all sewage treatment plants: For temporary camps or cantonments, average daily

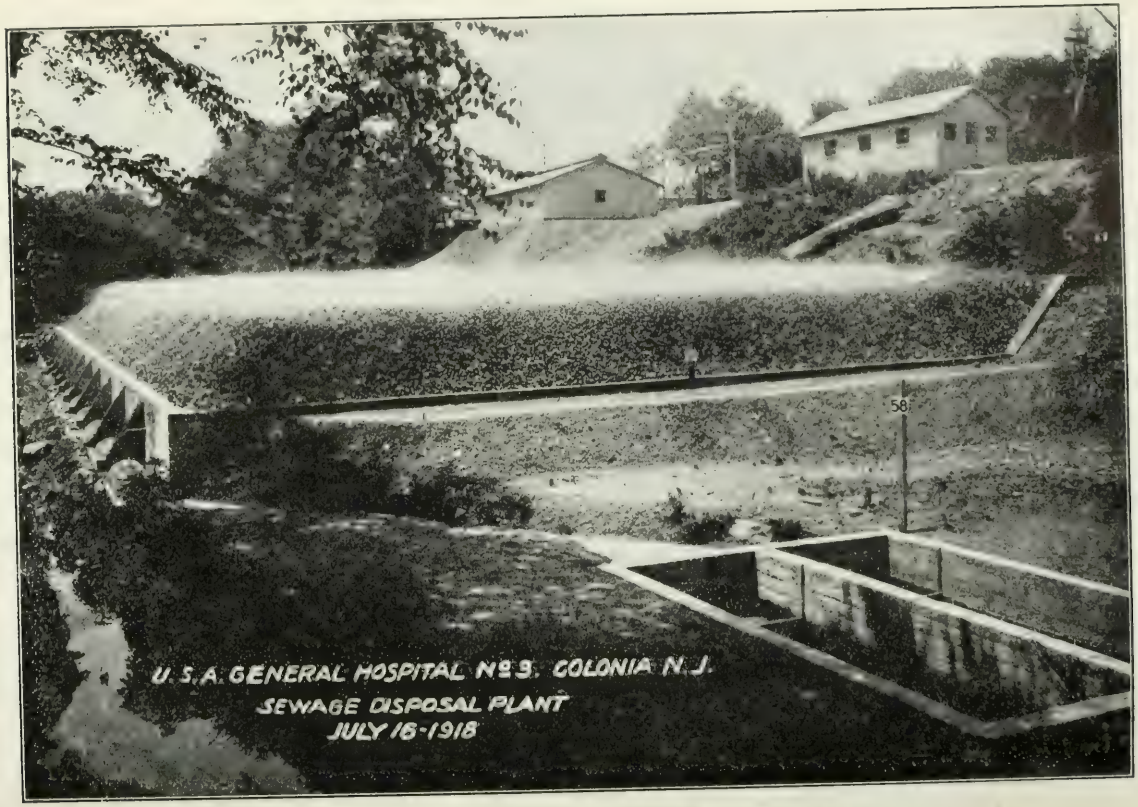


FIG. 5.—SEWAGE DISPOSAL PLANT AT U. S. A. GENERAL HOSPITAL NO. 3, AT COLONIA, N. J.

per capita flow of sewage, 60 gal.; hospital plants, 150 gal. per bed per day, and 60 gal. per attendant per day; average detention period in tanks, 6 hours, based on net capacity of tank, making a certain allowance for sludge capacity; maximum dosing period during highest rates of flow, 1½ min.; filter capacity, 30 000 population per acre, with 5-ft. depth.

Up to the close of 1918, the Construction Division has built 150 tanks of standard design and they have a total holding capacity of 20 000 000 gal. These plants are distributed throughout all parts of the United States. Comparatively few filters have been constructed, the total number being only 19, having a total area of 447 000 sq. ft.

The writer's experience leads to certain conclusions relative to sewage treatment plants and processes, which are summarized as follows:

Sewage contains both acid and base-forming constituents. If the base-forming elements predominate, the decomposition of sludge and scum will be rapid. If, on the other hand, the acid products of decomposition are not promptly neutralized, bacterial decomposition will be greatly retarded.

If the water portion of the sewage is high in alkalinity, the bacterial action will be vigorous, and the digestive capacity of a tank correspondingly increased. Co-ordination of certain observed facts relative to the performance of tanks in widely separated parts of the country has confirmed this conclusion.

The color of sludge alone is not a true indicator of thoroughness of digestion, as the sludge in an Imhoff tank may be blackened by the presence of iron sulphide, one of the products of decomposition, and still be incompletely digested. It is true, however, that well-digested sludge is either dark brown or black. Furthermore, the so-called "tarry" odor is not an essential characteristic of thoroughly digested sludge, as the writer has examined many samples which not only did not have an odor of this kind, but were practically odorless, and dried very rapidly, even under adverse conditions.

The effluent from a Construction Division type of tank, when functioning normally, is superior to that of a plain sedimentation tank, due to the fact that the very finely divided matter borne in suspension is partly mineralized.

The capacity of a trickling filter does not vary directly with the volume of filtering material, a 5-ft. filter giving as good results as a

deeper one. The average rate of dosing for filters of all depths should not exceed 2 000 000 gal. per acre per day, if a high degree of purification is desired. The short dosing period is of more importance than perfect distribution.

The humus discharged from properly functioning trickling filters should not create an objectionable condition due to putrefaction. The putrescible character of the effluents from some deep trickling filters, and also the humus held in suspension, is due either to an inferior quality of tank effluent or to improper dosing. If the flow from spray nozzles continues too long, the entire depth of filtering material will be drenched, with the result that insufficient time will be given for the thorough oxidation and nitrification of the organic matter. The flow from under-drains should be maintained at as uniform a rate as possible.

Treatment by intermittent sand filters, even in localities where the soil is of exceptionally good quality for the purpose, has not been as satisfactory as anticipated, owing to the unusually high grease content in the sewage. It has been necessary, consequently, in certain instances, to put in tanks for the preliminary treatment of the sewage before discharging it on the sand beds. Furthermore, such plants (intermittent sand filters) are usually higher in cost of construction, and considerably more expensive to operate, than those of the standard type.

The wide variations in rates of flow at camp sewage treatment plants, the extremely high suspended organic content of the sewage, and the inhibiting influence on bacterial development resulting from the excessively large quantities of grease, are important factors in determining the load which is borne by the camp sewage tanks. Statistics fail to show that there has ever been any instance where these adverse fundamental factors were so great. This is a fact that should not be overlooked in assigning tanks of this type to their proper place in the sanitary engineering practice of the future.

It is realized that the problem of sewage disposal is very complex, due to the great variety and combinations of waste matter which constitute sewage. There is an ample field for research work along several lines, in order to perfect the processes now in use, and to develop, if possible, plants which will give satisfactory results at lower construction cost. The economic factor should not be overlooked, as it has a most direct influence in determining the progress in this field of sanitary engineering.

DISCUSSION

Mr. Metcalf.

LEONARD METCALF,* M. AM. SOC. C. E.—This paper has summed up in a very good, descriptive way the admirable work done by the Construction Division. It is to be hoped that before Maj. Doten completes his work on this subject, he will submit with his paper some of the analytical results which have been made, bearing on the operation of the plants.

In judging of the work done in this branch of the service, as in the work of actually building the cantonments, one must remember the very short time which was available, in the first instance, for building the sewerage systems, as well as the camps.

The periods for which the camps were to be operated, was a very important factor in determining the most desirable kind of disposal.

Maj. Doten perhaps has not given as clear an idea of his worries in consequence of the grease problem, because the conditions, as he has pointed them out, have been found to be quite different from those realized in many municipal plants, and have been a source of very great difficulty.

The question of how to handle the grease problem in the best way, involved the question as to whether or not it would be possible, with so many different buildings, so many different centers in the camp, to separate the grease, as was desirable, before it mingled with the fœcal matter, because the grease would obviously be much more valuable, be more salable, if the separation could be made. The way in which this problem has been worked out seems to the speaker to be good.

Final judgment on it cannot yet be passed, perhaps, but certainly the underlying principles—to have a large area for cooling the hot liquids, the tubs of dish-water that are turned into the sink, which on account of their volume and temperature tend to carry the grease forward; the small outlet, which will give scouring velocity; and sufficient slopes to enable the traps to clear themselves of solid matter—are all sound. Although some changes may be necessary perhaps, a definite step in advance seems to have been made in the matter of the economical handling of the grease problem in camps and cantonments.

There is also included in this grease problem the question of good administration. It has been felt very markedly in these cantonments, as well as in municipal plants, and is a factor which has to be kept in mind by the designing engineer when he comes to pass on the details of his plant, because it is possible in military camps to give orders and to have them enforced in a way which is not always possible in municipal plants.

The acid or alkaline character of the sewage solids in the tanks varies with the character of the water supply, and affects the sludge digestion.

* Boston, Mass.

Mr. Metcalf.

The best design for the tank which Maj. Doten has shown, and its efficiency as a digestion tank, is still open to debate and proof. It is a question on which various men who have observed these tanks, and some who have made experiments on them, perhaps differ; and for that reason it is desirable for Maj. Doten to submit the facts, as he has been able to develop them, with the greater facilities which he has had recently for following by chemical analysis in a general way the operation of these tanks.

It has seemed to the speaker, as he has visited and viewed perhaps twelve or fifteen of the camps, in the West, in the South, and around New York City, though he has not had the opportunity recently to keep closely in touch with them, that the tank was more essentially a clarification than a digestion tank, and that its most effective operation was found in its use as a sedimentation tank rather than as a digestion tank. This was anticipated in principle by some, and, it is believed, has been proved in operation.

On that question Maj. Doten may differ radically with the speaker, and that is one reason for hoping that he will give the facts, in order that the truth may be established.

In the early operation of the tanks, it is to be borne in mind that the available capacity was substantially greater than later, when the numbers of men at the camps had increased. The tanks were designed on the basis of 10 gal. per capita. On the basis of that capacity, experience seems to have demonstrated that they could not act as true digestion tanks. In other words, when the attempt was made to operate them as digestion as well as sedimentation tanks, they were soon gorged, and the accumulated solids had to be removed. Sedimentation was their best function, under those circumstances, and, if so, it is necessary to limit the accumulation of solids in the first compartment of the tank by removal of the excess of sludge and scum in order that the digestion process can go on in the other compartments. If solids are removed, as has been necessary, the question immediately arises, what is the relative percentage of the solids disposed of by digestion, on the one hand, and by sedimentation and removal in an undigested state, on the other.

The effect of the different methods of operation on the freshness or staleness of the tank effluent is also of much importance. As the process of removal is not a pleasant one, it is to be avoided as far as practicable.

Good judgment was shown in the adoption of the 10 gal. per capita basis of design, chosen as an economical procedure, for the reason that the life of the camps, and hence of the disposal works, was limited to a period of 5 years, and that it was recognized that it might not be greater than 2 years. That has proved to be the case in the majority of the camps. The lower first cost of sedimentation tanks was balanced against the lower operating expenses of digestion tanks.

Mr. Metcalf.

The speaker questions whether the form of the pyramidal bottom was worth its cost, as observed at the various camps. There is no doubt that the tank with a pyramidal bottom was materially more expensive than a rectangular tank of the same capacity. It may be questioned whether in operation it was any more economical, because the slopes are not steep enough to force the solids down. On the other hand, they are so steep as to be impossible to stand on for scraping, though they can be flushed by hose.

The speaker is very much surprised at Maj. Doten's statement in regard to the efficiency of tanks of various depths, because this appears to be out of line with the results of experiment, and of experience in municipal plants in the United States which have been operated up to this time, so far as the speaker knows them. It is to be presumed that Maj. Doten speaks with knowledge derived from his experience, and, therefore, his evidence will be awaited with interest.

In the matter of placing the sprinkler nozzles on the trickling filter beds on a slope, that is, not in the same plane, the speaker's observation has indicated it to be distinctly disadvantageous rather than advantageous, as it has resulted in overtaxing locally certain portions of the bed, and has failed to give as good distribution as under the more usual course of procedure.

In so far as municipal practice is concerned, some other types of under-drains will probably prove more advantageous than the form which has been used in these camps; nevertheless, good judgment was shown by Maj. Doten in using the simplest form, in the interest of greater economy for temporary construction. Maj. Doten's record of service is much to his credit.

Mr. Hill.

NICHOLAS S. HILL, JR.,* M. AM. SOC. C. E.—Mr. Metcalf has covered some of the points which the speaker had in mind to make. He wishes, however, to emphasize the statement that these tanks are primarily clarification tanks, and not digestion tanks. From his connection with their operation, the speaker feels that a saving in first cost could have been effected, and with as satisfactory results, if their design had been simplified. He is satisfied that the division into compartments was a mistake. At Camp Merritt it was found that the major part of the sedimentation occurred in the first compartment, and that the second, third, and fourth compartments contained only a small quantity of sludge. In fact, at Camp Merritt, a by-pass was provided from the first tank, and arranged so that sludge could be transferred therefrom to the second, third, and fourth compartments. If this had not been done, the first compartment would have had to be cleaned frequently, and it would have been necessary to clean the remaining compartments at intervals, increasing in length, up to a maximum for the last compartment.

* New York City.

Mr. Hill. The speaker has serious doubts about the design of the grease traps adopted, and whether the high velocity idea is a practical one. He would not design a grease trap along the lines of the one used for the cantonments, and, from observation of the operation of these traps, he knows that there was lost from them a large quantity of grease which could have been saved with slight modifications of design.

In the camp with which the speaker had to do, difficulty was found in getting those who attended to the grease traps to remove the grease from the surface around them. The result was that with grease lying on the surface in the summer, an attractive center for flies was created, which was objectionable, particularly as the traps were near the kitchens. It is realized that it was necessary to remove the grease at the source of production, before it mingled with human wastes, if it was to be obtained in a condition ready for use, but the system of placing individual grease traps on kitchen connections in municipalities would be impracticable, for various reasons, and, if grease is to be withdrawn from the sewage, it will have to be done at the point where the sewage is treated. The speaker mentions this for fear that some one, reading of the successful operation of the grease traps in the cantonments, will seize on the idea that a large return can be secured in municipalities by adopting the same process. If individual grease traps can be adapted to municipal work, their form will certainly have to be modified materially from that used at the cantonments.

The general design for the sewerage systems for the cantonments showed much foresight on the part of those who laid them out, and the assumptions which were made were borne out in general by actual experience in the several camps. The design generally is to be commended.

In connection with the work of construction, it might be well to refer to the use of trenching machines. The conditions at Camp Merritt were such that the use of these machines was practically impossible. The contractors procured ten or twelve of them, with the idea of doing the work expeditiously, but when the trenches were opened, rock was found in some cases, and in other cases the sandy soil contained water, and where that was the case, it was found that hand labor was more economical than the trenching machine, because it was practically impossible to sheathe the trench directly behind the machine. The trench caved in, and the hand work required in removing the fallen material was almost equivalent to that which had to be done originally. A number of methods of working behind these trenching machines in wet ditches was tried without success. They may be modified so that they can be used where the trenches are wet, but, as designed to-day, they are not useful under such conditions. These machines were handicapped to such an extent in various places, on account of obstructions, that hand work had to be resorted to for a

large proportion of the work, and, at Camp Merritt, hand labor was found to be more expeditious and cheaper. Of course, where the trenches were shallow, as in the case of water mains, a small trenching machine was used to great advantage. The difficulties referred to relate more particularly to the use of these machines for sewer work. Mr. Hill.

ALMON L. FALES,* M. AM. SOC. C. E.—This paper is especially interesting to the speaker, because of his work for the Construction Division of the Army, in connection with water supply and sewage treatment at the army camps. He has inspected the sewage tanks, designed by Maj. Doten, in operation at numerous camps, and has made thorough investigation of their efficiency at several of the camps. Mr. Fales.

The efficiency of sewage tanks may be judged by three important standards:

1.—Removal of suspended solids in the sewage, which will settle to the bottom as sludge or rise to the top as scum;
2.—Digestion of the solids removed, with the production of inoffensive matter.
3.—Reduction in biochemical oxygen demand of the sewage, measured by the relative volumes of diluting water necessary to render the sewage and tank effluent non-putrefactive.

Judged by the first and third of these standards, these tanks were found to be very efficient, when the accumulation of sludge and scum was removed with sufficient frequency to avoid excessive fermentation or septic action and to prevent disgorgement of solids at times of high flow. In other words, they were efficient when operated as plain sedimentation tanks. The nature of the fresh uncomminuted camp sewage is favorable to the removal of a large proportion of the suspended solids by sedimentation and flotation.

Judged by the second standard—digestion of solids—these tanks were not efficient, with a capacity of 10 gal. per capita served, which was the basis of design; and tests at one camp indicated that complete digestion of the solids could not be secured even with a capacity as high as 35 gal. per capita. At several camps some well-digested sludge was produced in the third and fourth compartments, due, in part at least, to the mechanical separation of the less easily digested solids in the first and second compartments, but, as far as the speaker is aware, in no case was the rate of production of well-digested sludge equal to the rate of accumulation of fresh solids in the tank.

To what extent the inefficiency of the tanks in the digestion of solids was due to the large proportion of fats in the camp sewage has not been ascertained. There is no doubt that some improvement will result from placing efficient grease traps on the kitchen drains. With

* Boston, Mass.

Mr. Fales.

such traps it seems certain that satisfactory digestion of the solids can be secured, if unlimited tank area and capacity are available. The area and capacity required in the tanks to accomplish this have not been determined, but they exceed by far the basis of design.

If the tanks are made adequate for the digestion of the solids, and are operated according to the principle of the designer, the tank effluent will be charged with the gaseous and liquid products of digestion and with more or less finely-divided suspended and colloidal matter, resulting from the process of digestion. The effect of this action is to render the tanks inefficient, judged by the third standard, namely, the reduction in the biochemical oxygen demand of the sewage. At some camps, where digestion was taking place, the tank effluent was found to have a higher oxygen demand than the crude sewage itself. Such a tank effluent is likely to be malodorous and to require an abnormally large proportion of chlorine for disinfection and a relatively large volume of diluting water to render it non-putrescible. Further, it is the consensus of opinion that such a tank effluent cannot be treated by processes of bacterial oxidation—such as filtration through trickling filters—as advantageously as one which is not affected by the products of decomposition of the sewage solids.

In the speaker's opinion, to secure efficient tank treatment it is necessary to separate the functions of sedimentation and sludge digestion. This may be accomplished by the so-called two-story tank, in which the opening from the sedimentation compartment into the sludge compartment is trapped; by separate sludge digestion tanks; or by operating tank units in rotation, first for sedimentation and then for sludge digestion.

In accordance with the recommendation of the consulting engineers (Messrs. Fuller and McClintock, and Metcalf and Eddy), the number of tank units, at most of the large camps, has been, or is being, increased to at least five of 7.5 gal. per capita each, in order to permit of operation by the rotation method; and provision is made for pumping the sludge and scum from any compartment of any tank to any other compartment, in order to facilitate the digestion of the solids.

The speaker is glad to corroborate what Maj. Doten has said in regard to the character of the effluent from the sedimentation tanks at the Great Lakes Naval Station.

At the time the speaker was there, this effluent was characteristic of that from septic tanks—black and foul and heavily charged with fine suspended matter, probably to a large extent sulphide of iron. In answer to a question raised by the speaker, he was informed that it was out of the question to allow this effluent to flow down the aerating effluent flume, on account of the foul odors created.

This condition of the effluent was evidently due to the presence of decomposing sludge in the sedimentation tanks, and illustrates the

effect of attempting to secure sedimentation and sludge digestion in the same tank, at the same time. Mr. Fales.

Whether the sludge was not being drawn from the sedimentation tanks into the separate sludge digestion tanks with sufficient frequency, or whether the bottom slopes of the sedimentation tanks were not steep enough to be self-cleansing, and thereby prevent accumulations of solids in the tanks, was not ascertained.

KENNETH ALLEN,* M. AM. SOC. C. E.—It might be of interest to mention a tank devised by J. W. Alvord, M. Am. Soc. C. E., and used as a standard by the United States Housing Corporation. Mr. Allen.

The unit (Fig. 6) is supposed to serve for a 1 000-house development. It consists of a primary settling tank with three hopper bottoms 12 ft. square, sloping 60° from the horizontal, and extending about 16 or 17 ft. below the top of the tank; a secondary settling tank, similar and parallel to the one described; and, between these two, a sludge digestion tank of equal length, width, and depth, but with a single hopper.

The baffled sewage flows from the primary to the secondary tank over the sludge tank by several troughs, and the scum can be pushed along special troughs connecting each settling tank with the sludge tank.

The sludge which collects in the six hoppers of the primary and secondary tanks is drawn into the hopper of the sludge tank merely by lowering the surface of the liquid in the latter a foot or so, is retained there until digested, and is then drawn off by a sludge pipe in the bottom.

With a few feet of fall to the sludge bed, there is seen to be no necessity for pumping the sludge, the movement of which is controlled by valves.

The tank is designed especially for fresh domestic sewage, such as is produced in small towns or cantonments, with an effluent, after a detention of from 30 to 50 min., still relatively fresh and containing dissolved oxygen. To obviate the difficult and objectionable removal and disposal of scum by hand, this is readily disposed of with the sludge. A further advantage lies in avoiding the excessively deep excavation usually necessary with the Imhoff tank.

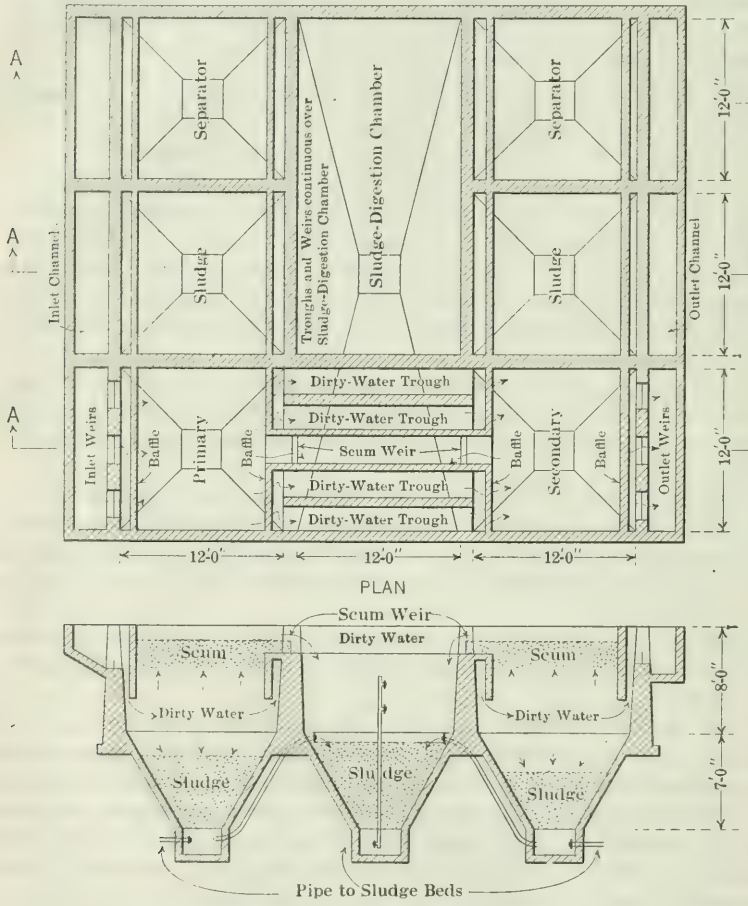
A tank of this description has been in continuous operation at the Great Lakes Naval Training Station since October, 1917. It consists of three units, each designed to handle 1 000 000 gal. per day, with a detention period of 45 min. in each tank. It is understood that there has been no appreciable accumulation of sludge at this plant, and that it has been very successful in operation. The small quantity of sludge has been attributed to an increased fermentation due to

* New York City.

Mr. Allen.

an occasional seeding with a wisp of hay, but this is not believed to be proved as yet.

The speaker does not know what difference there is between the character of the sewage at these works and that at the cantonments.



TYPICAL POSITIVE SEWAGE SEPARATOR AND DIGESTOR
FOR FRESH SEWAGE
1000-HOUSE DEVELOPMENT
FOLLOWING PRINCIPLES PROPOSED BY JOHN W. ALVORD - CONSULTING ENGINEER

FIG. 6.

The plants of the Housing Corporation have not been operating long enough to reach a determination, but that at the Great Lakes Station has been in use for more than a year, and, thus far, seems to have given good results.

Mr. Allen.

With reference to the Great Lakes plant, the following additional information has been received from L. R. Howson, Assoc. M. Am. Soc. C. E., who has been intimately connected with its design and operation:

"The writer visited the Great Lakes Station on Thursday of this week—the day before your letter was received.

"The tanks have been practically free from odor, even in the immediate vicinity of the plant, and entirely free from odor at a distance as great as 50 or 100 ft.

"The sewage is very fresh, the age of the sewage when it reaches the plant being from 5 min. to a little over an hour from point of origin to the time it reaches the plant. The average age is less than 45 min. This and its purely domestic character account for the formation of great quantities of scum and very little sludge. The scum forms to a depth of from 2 to 3 ft. per week over the primary tanks.

"The tanks have operated very successfully, and an average of approximately 80% of the suspended matter has been removed by these tanks alone.

"When the tanks were first placed in operation a strong hydrogen sulphide odor was observed. The chemist at the Naval Training Station did considerable experimental work to overcome this odor, and worked out some interesting data with regard to liquefying this sludge and scum.

"The sludge digestion chamber was also seeded with liquefying bacteria, with the result that the plant has now been in operation for a year and a half, operating the greater part of the time at approximately 50% greater rate than that for which it was designed, and, although the sludge has been discharged into the central digestion chamber at weekly intervals, it has not been necessary to draw off any sludge to the sludge beds to date, although the comparatively clear liquid resulting from the liquefying of the sludge has necessarily been disposed of.

"There have been some difficulties observed in the operation of the plant as a whole, which are not, however, attributable to the tanks. We had some 40 ft. of head to spare between the tanks, which are on the bluff, and the filtration plant at the foot of the bluff. This was utilized by aerating steps, which we considered and showed on our plans as being desirable for use at certain seasons of the year and when the wind was off-shore. The base hospital with its additions are within 300 ft. of the sewage disposal plant, and it must therefore operate entirely free from odor at such time as the wind is blowing on shore and from the plant toward the hospital buildings. The only complaints from odors which we have observed have been due to the use of the aerating steps, and these have been very slight. The use of the steps is now discontinued, and they will not be used except during favorable wind conditions."

Plants of this type have been built at Madison, Wis., Aberdeen, Md., Bethlehem, Pa. and Swedeland, Pa.

Mr. Kershaw.

G. Bertram Kershaw,* M. Am. Soc. C. E. (by letter).—This paper brings out the marked difference between municipal and camp sewages,

* London, England.

Mr. Kershaw.

the latter not only being considerably stronger, but, in addition, containing considerable grease.

It may be of interest, for purposes of comparison, to give some figures of analysis for an ordinary domestic sewage and for a barracks sewage, the latter in Great Britain differing but litttle from camp sewage. The following figures relate to the average of three sets of hourly samples, taken at the Aldershot Camp Sewage Farm:

	Parts per 100 000.	Parts per 1 000 000.
Ammoniacal nitrogen	7.87	78.7
Total organic nitrogen	5.06	50.6
Albumenoid nitrogen	1.62	16.2
Total nitrogen	13.3	133.
Oxygen absorbed in 4 hours	20.79	207.9
Chlorine	14.98	149.8
Suspended solids	36.6	366.

The Caterham Barracks figures were obtained from the average of four sets of hourly samples, and are even stronger:

	Parts per 100 000.	Parts per 1 000 000.
Ammoniacal nitrogen	31.41	314.1
Albumenoid nitrogen	1.75	17.5
Total organic nitrogen	3.87	38.7
Total nitrogen	17.53	175.3
Oxygen absorbed in 4 hours	17.99	179.9
Chlorine	13.57	135.7
Suspended solids	42.10	421.

The sewage of Kingston-on-Thames may be regarded as an average domestic sewage, and the figures are also obtained from sets of hourly samples, drawn according to rate of flow.

	Parts per 100 000.	Parts per 1 000 000.
Ammoniacal nitrogen	3.85	38.5
Albumenoid nitrogen	0.86	8.6
Total organic nitrogen	1.63	16.3
Total nitrogen	5.49	54.9
Oxygen absorbed in 4 hours	10.66	106.6
Chlorine	9.43	94.3
Suspended solids	31.	310.

The foregoing figures demonstrate the strength in nitrogenous matter of camp sewages as compared with domestic sewages, and this is due no doubt to the fact that, as the author points out, the population is adult, and, further, the quantity of nitrogenous food consumed is large. The same thing has been observed in Great Britain, in the case of colliery towns and villages; in these cases, however, the water supply is often low.

Mr. Kershaw.

During the first stage of the war, considerable difficulty was experienced here with grease, and specially large grease traps were put into use for intercepting it. Disinfectants, too, were often used in sufficient quantity to interfere materially with biological purification. Grease, however, was the chief difficulty, and, as might have been anticipated, it caused trouble at all stages of treatment, while it was some little time before steps were taken to utilize the grease recovered.

In the early stages of the war, many of the camp sewage treatment works left a great deal to be desired, both as regards design, and ability to cope with the very concentrated sewages. Moreover, in some instances, several small treatment works were constructed, where, with a little ordinary engineering skill, one plant would have sufficed.

In the camps in Great Britain, it was found that the sewage flow was almost invariably divided into distinct flushes, the flow being practically confined to about from 10 to 12 hours out of the 24. Consequently, when the filter capacity was calculated on the basis of the sewage flow of an ordinary town, at a rate of, say, 75 gal. per cu. yd. per 24 hours, it was about 50% too small. This fact was often lost sight of at first.

With respect to water supply per capita, the figures given by the military authorities were usually found to be much too low, and 30 gal. per head was determined as being about right, in fact, it was the usual figure taken for a town water supply.

With regard to manholes, it is the customary rule in England, that the distance apart shall not be in excess of about 300 ft. Some years ago, lampholes were permitted between the manholes, but, lately, they have been discouraged by the Local Government Board. In the case of the majority of the camps, full treatment by tanks and filters has been carried out, as conditions were seldom such as to warrant the discharge of tank liquors without further treatment.

The following details of a plant designed to deal with a total sewage flow of about 20 000 gal. per day, may be instructive, as showing that the treatment of a strong camp sewage is by no means a simple matter:

1. Grease trap.
2. Covered septic tank, capacity about 15 000 gal., with inverted pyramid bottom for removal of sludge.
3. Small mixing tank for chemicals.
4. Four fill and draw settling tanks, capacity of each about 5 000 gal., with floating arms and sludge valves.
5. Two dosing tanks, capacity of each about 200 gal.
6. Two deep percolating filters of coarse coke or clinker, capacity of each about 250 cu. yd., with automatic distributors.
7. Effluent settling tank, capacity about 2 000 gal., with inverted pyramid bottom for removal of humus.

Mr. Kershaw.

8. General sludge sump, provided with chain-pump, and communicating with septic tank, settling tanks, and effluent settling tank.
9. Two shallow effluent straining filters of fine coke or clinker, each about 20 sq. yd. in area and 9 in. deep.
10. An area of land set aside for trenching the sludge, which would be considerable in volume.

This is a specification written by Maj. C. C. Frye, one of the chemists to the recent Royal Commission on Sewage Disposal.

The writer is unable to follow the author's statement that, "the capacity of a trickling filter does not vary directly with the volume of filtering material, a 5-ft. filter giving as good results as a deeper one." Surely this depends very largely on the size and character of the filtering material used; in other words, on the duration of contact of the liquor with the filtering material.

Comparison of the work done even by similar filters, with similar distribution, is beset with pitfalls. For example, a 5-ft. filter (*A*), may be working below its full power and producing a good effluent at a certain given rate of filtration per cubic yard; it would be erroneous to dose a 10-ft. filter (*B*), at a rate three times that of Filter *A*, and, because it produces an effluent similar to that of Filter *A*, to assume that Filter *B* can treat three times as much as Filter *A*, as Filter *A* is not working up to its full capacity; this destroys any comparison.

Concerning continuous and intermittent distribution on filters, it is to be observed that if the tank liquor is applied to filters not continuously, but in flushes, this must result—other things being equal—in increasing the volume treated per cubic yard in a given time. It would seem to be most desirable, in comparisons of treatment plants, to give the numerical figure of strength of sewage dealt with in each case.

Mr. Besselievre.

Edmund B. Besselievre,* Assoc. M. Am. Soc. C. E. (by letter).—The subject-matter of this paper is of great interest to the writer as he has specialized in sewage treatment, and, therefore, it possesses a peculiar claim on his attention. He also had the opportunity of being associated with the author, during the last year of the war, in the solution of sanitary problems at the various camps and cantonments of the United States Army.

The salient points of the paper which appeal to the writer, and which it is his intention to discuss, are the striking variations in composition between ordinary municipal sewages and those of the camps; the conditions entering into the selection and development of a proper type of tank for treating this sewage; the need for careful design of

* Washington, D. C.

tanks and auxiliary units; and the necessity for careful operation of the tanks and co-operation between the designer and the operator. Mr. Besselievre.

The writer feels that the author has covered, clearly and satisfactorily, the question of reasons for adopting the single-story tank instead of the type which had been most commonly used by nearly all civilian engineers previous to the war. It is only fair to state here that previous to the writer's association with the Construction Division, he felt, as did the majority of sanitary experts, that the two-story, or Imhoff, tank was productive of the best results for all municipal sewage problems. However, a careful consideration of the subject and a thorough understanding, gained by personal investigation and observation at various camps and the study of reports of results, etc., has changed this opinion to a great extent.

The conditions peculiar to the production of camp sewage were so different from those of ordinary municipal work that, considering these conditions and the need for speedy construction and economy in cost, it was seen that a single-story tank with some modifications from the older types of the septic tank would be the most reasonable one to adopt.

The life and habits of the soldier are regulated by strict discipline, producing a great tendency toward unusual concentration, at certain short periods, of the usage of sanitary facilities. This established a peak load in the sewers higher in all cases than in those of any municipality of which the writer could find record; and, also, due to the controlled usage of water, produced a high concentration of the sewage itself. Owing to the congested housing conditions and comparatively short length of run to the point of tankage, the result was a fresh, practically uncomminuted sewage, with a high suspended organic solid content.

Naturally, in comparison with a sewage traveling a relatively long distance and one that is more dilute and, due to length of flow, highly comminuted and partly septic, the first mentioned, or camp, sewage would require, regardless of any consideration of the composition of the diluting water, a tankage of greater duration in order to allow of proper and sufficient deposition of suspended and settleable solids during the passage through, and greater provision for the retention and digestion of the resultant deposit. These were the basic principles of the tanks designed and built by the Construction Division, under the author's direction.

The writer was greatly astonished when, during a study of the constituents of municipal sewages and those of various camps, the tremendously high, suspended, organic solid content of the camp sewages was made apparent. The data on page 343, showing averages only, do not indicate the maximum; but with an average of 750 parts

Mr. Besselievre.

per million of suspended organic solids, it can be easily assumed that the maximum was considerably higher.

As this average is so much greater than that for any municipal sewage of which the writer has had reports, it does not require, in his opinion, any further arguments to show the necessity, to any sanitary engineer, for a tank different from the usual municipal type, disregarding for the moment the considerations of speed of construction, cost, and the temporary character of the camps.

In either camp or municipal practice mere excellence of design does not necessarily guarantee satisfactory results. To the writer fully 90% of the production of a satisfactory, unoffensive tank effluent depends on the intelligent operation of the unit, based on the recommendations of the designer and a thorough understanding of the principles involved in each particular design.

Unfortunately, in quite a few cases, the tanks built by the Construction Division, despite all precautions and recommendations, did not have the co-operation of operator and designer and, due to this, there were several instances where, in two places, similar tanks working with sewage of like character gave results nearly diametrically opposite.

During the past 13 years, the writer has visited professionally a majority of the sewage treatment plants in the United States, and observations at these places have strongly embedded the conviction that the principal cause of failure in the production of satisfactory results is due to faulty operation.

Excellent design, embodying careful consideration of all conditions, has been negatived in several cases by the lack of intelligent operation or intentional neglect, due either to want of interest or knowledge on the part of the municipal authorities, or to the natural stupidity or laziness of the operator who has been selected in all probability for political reasons rather than for his ability to operate the plant as intended. On the other hand, plants improperly designed were, nevertheless, due to careful and intelligent operation, capable of producing the results intended at the time of design.

Despite the military discipline at the army camps, it was not possible, in all cases, to obtain the degree of control desired by the engineers of the Construction Division. Lack of knowledge of the principles involved in the design or proper operation, coupled with the sad fact that the average layman considers, usually erroneously, that a sewage disposal plant is a disagreeable place in which to work, and the supposition that intentionally faulty operation would secure release from such a detail, may have been the causes contributing to the trouble in the few cases in which it arose.

The writer is a strong advocate, in all cases where sewage treatment plants are built, of having it distinctly a part of the engineer's contract that not only is he to design and supervise the construction of the

Mr. Besselievre.

plant, but he is to have the selection of the attendant and be responsible for his proper instruction. He should make periodical inspection trips to the plant for a certain specified time in order to ascertain whether all instructions are being carried out and the desired results obtained.

In the field of plant operation and control, the State of New Jersey has undoubtedly taken a long step forward by the passage of a license law for operators of sewage disposal plants. This law provides for intelligent operation, and the unique Sewage Works Association of New Jersey has been one of strongest advocates of such legislation and has done more than any other organization to bring it about and thereby place New Jersey as a pioneer in this phase of engineering, the intelligent operation of public utilities. It would be well if other States followed her example in this regard.

In the determination of the retardent action of grease on proper septic tank action and sludge digestion and the formation of non-soluble scums, a long step forward was taken by the adoption of proper grease-retaining traps, either at some point near the septic tank or, as in the later work, by separate traps at each kitchen or group of kitchens. The results obtained in the operation of tanks and filters after the removal of the grease content, have more than justified the opinion of the detrimental action of grease, and their cost has been a minor item compared with the beneficial results and the reclamation value of the grease obtained from these traps before its pollution by sewage.

Instances have been known where a tank which had been operating indifferently before the removal of the grease, would show entirely different and eminently more satisfactory results, after it had been thoroughly cleaned and the grease traps introduced. In view of the results obtained, the writer feels that, for the sake of obtaining better and more rapid sludge digestion, freedom from scum, and the protection of sprinkling filters, some form of grease-retaining device should be introduced, regardless of the reclamation value of the grease obtained.

It is, of course, manifestly impracticable in municipal work to introduce a grease trap in each house, as this would make necessary the construction of some central grease-retention device. This would eliminate any reclamation value of the grease, as it would be polluted by sewage and, therefore, worthless, but the increase in efficiency of the tank would more than offset the slight additional expense of the trap.

In one case with which the writer is familiar, namely, a camp in the District of Columbia, the average weekly total accumulation of grease removed from the traps in the camp, which was a small one, was 2 000 lb.; and after a small soap-making plant had been set up, the authorities were able to make sufficient soap from this grease to supply all the kitchens and laundries in the camp.

Mr. Besselievre.

In line with the author's call for further research and development work with a view to the procurement of more satisfactory plants, the writer desires to elaborate on several features of design, which, not having been given the necessary study, have caused trouble in a number of municipal plants inspected by him.

In referring to the design of units of a sewage treatment plant, the writer wishes to call attention to a particular feature in connection with the design of dosing tanks for sprinkling filters, which has been brought strongly to his attention by visits to more than 150 municipal plants in the United States and the observation of their failure to function properly due to lack of proper attention to this detail.

Although the dosing tank is a small unit of a sewage treatment plant, and is, the writer fears, one that is usually passed over hurriedly in design, it is, nevertheless, one of the most important units, and unless proper and intelligent design is made, an otherwise well designed plant can be made very inefficient.

If the dosing tank is not properly proportioned to the volume of flow to be handled, the filter may be improperly dosed and a negative result secured in the filter effluent.

Regardless of the length of the dosing period, the capacity of the tank, its shape, and general features of construction, the method of connection to the filter distribution piping should be given careful attention and study.

The size of the tank should be proportioned to the dosing period and the inflow, so that the volume discharged during the total discharge period will not be excessive during abnormal flows. Therefore, a careful selection of the dosing apparatus should be made so as to provide for the drawing down and "breaking" of the siphon action at the predetermined low-water line, regardless of the rate of inflow.

The writer has seen numerous municipal plants wherein this point has not been carefully considered and, due to insufficient siphon size or nozzle capacity, the siphon at times of maximum flow would go into continuous operation, with the consequent flooding of the filter for periods of hours. This results in canalization of the effluent through the filter material and the production of negative results as to purification of the effluent. A careful study of this point, with a proper use of the data provided by the manufacturers of siphons and nozzles, will tend to eliminate the trouble which may be caused very easily by neglect of this particular feature.

Another important feature in the design of a sewage treatment plant, and one which although not exactly in the dosing tank, nevertheless applies to it, is the consideration of the means of connection between the siphon mouth and the distribution system of the filter. Neglect of the importance of this point and improper design has caused, to the writer's knowledge, the failure of several municipal plants.

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Failure to design this connecting link properly has resulted usually in the loss of the most valuable head in the dosing tank, namely, the first few inches or first foot. Such loss is due to laying the pipe connecting the siphon mouth with the distribution system above the level of the nozzle orifices so that after each discharge of the siphon this piping was drained and at the next discharge, sufficient liquid was required to fill this empty piping before the water would appear at the nozzles. This was sufficient to reduce the head in the dosing tank to such an extent that the radius of nozzle spray, in turn, was reduced to a few feet from the nozzle, leaving the filter with a very large percentage of unwetted area.

The effect of such a small radius spray is an overdosing of the portion of the filter immediately adjacent to each nozzle and tendency to canalization of the liquid through the filter without proper bacterial action.

In one particular municipal plant, with which the writer is familiar, the loss occasioned by this feature was such that with a dosing tank having a draft of 24 in., the loss of head, due to improper layout of piping, was 10 in., and the resultant radius of spray was about 4 to 5 ft., whereas the design of the plant was based on a radius of 11 to 12 ft.

In all cases, the discharge piping after leaving the siphon mouth should be dropped at once to such a level that the top of the pipe will be below the elevation of the lowest nozzle on the filter bed. It is impossible, of course, to get all the piping below the nozzle orifice, but with the drop beginning immediately after the vent connection at the siphon mouth, the minimum amount of piping will be emptied; the volume of this piping may then be ascertained and allowance made in the design of the tank so that the first inch or two from the top will contain sufficient liquid to fill this empty piping without the loss of the most valuable head of all, the initial head.

This consideration of the layout of the piping between the siphon and the filter is all the more important when a long period of dosing is used, with a corresponding long period of filling, during which period the piping is draining. With a short dosing period and a short filling period, the draining period is of necessity also shortened, and the emptying of the pipe lessened, so that the harm in this case is reduced; but, nevertheless, regardless of the length of the dosing period, this point should be carefully considered.

In closing, the writer desires to state that he believes that the type of tank and plants described by the author, for conditions similar to those found in army camps, when properly operated in accordance with the ideas and opinions of the designer, have given eminently satisfactory results.

Mr. Hammond.

GEORGE T. HAMMOND,* M. AM. SOC. C. E. (by letter).—This paper should prove of much interest to the Engineering Profession. It sug-

* Brooklyn, N. Y.

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gests the way to simpler methods of sewage and wastes disposal, and to further research work along certain practical lines. It also gives an interesting insight into some of the sanitary problems which had to be solved, under much stress and haste, by the engineers of the United States Army.

During 1918, and the first part of 1919, the writer was afforded an excellent opportunity to study, at first hand, some of the structures and methods which are described in this paper. He was not only able to make numerous visits to many of the cantonments and camps, but, in the course of his duties as an engineer, he was called on to design and erect several similar structures and to observe their operation.

During the period previously mentioned, the writer was in the service of the United States Government, at first as Sewage Disposal Expert and Sanitary Engineer to the United States Shipping Board, Emergency Fleet Corporation, on the staff of Morris Knowles, M. Am. Soc. C. E., Chief Engineer, Passenger Transportation and Housing Department, and had charge of the preparation of standard specifications for, and reports on, the sewage disposal of the industrial housing projects, as well as the review of plans for such structures. In many instances, these projects were large towns, some of which contained more than 2 000 houses. Like the cantonments, they had to be built in the utmost haste, and some of the problems met were quite similar, though on a smaller scale. Similar questions in sewage disposal arose, and, in several instances, the Doten plans for disposal plants and for grease traps were adopted. As Consulting Engineer, the writer has recently been in touch with these plants and is able to state that they are giving satisfactory service.

Following his service with the Housing Department, the writer was assigned to the staff of Lt.-Col. P. S. Doane, U. S. A., Director of Health and Sanitation of the U. S. Shipping Board, and was placed in charge, as Sanitary Engineer and Health Officer, of District No. 2, extending from Providence, R. I., to Camden, N. J., and containing thirty-five shipyards. Later, this department was by order of President Wilson, transferred to the U. S. Public Health Service, Assistant Surgeon-Gen. L. L. Lumsden succeeding Lt.-Col. Doane, as Director, under whom the writer continued to serve. While thus engaged, he had to provide a number of structures very similar to those described in this paper.

The writer came to regard the sanitary problems involved from much the same standpoint as the author, and to follow about the same methods in their solution, especially as certain designs and structures adopted under emergency conditions had proved, with properly regulated service, to be more successful than was expected and could be constructed quickly and cheaply.

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The sanitary requirements met in the shipyards, and in the housing projects connected with them, arose mainly from the great number of men suddenly brought together in places where all utilities were absent, until provided, for housing or feeding them, or otherwise caring for them. The shipyards sprang up like mushrooms wherever a ship could be built and launched, often at places quite distant from cities; and the progress of these yards was quite wonderful. Where a few hundred men were at work one day, thousands might be working within a week; probably more than 500 000 men were engaged at one time in shipbuilding.

In the shipyards of District No. 2, there was at one time an aggregate of more than 100 000 men. In several yards, there were more than 10 000 men each, and in one more than 15 000. Dormitories, mess halls, restaurants, and other houses had to be erected, with the attendant sanitary arrangements, and water supply, sewers, and sewage disposal facilities had to be provided in haste.

The paper discloses only a meager outline of the difficulties overcome by the War Department, in that hurried rush to put our military house in order. Much might be said about it. Those who saw this effort under way will never forget what a tremendous achievement it was, or how well the organization created for this great service succeeded in its work. Yet, it is also true that the undertaking was even more vast than was realized by those engaged in it. We should never again permit our country to be caught in such an unprepared condition. The results achieved in getting ready our army, our ordnance, and our ships, seem scarcely less than marvelous. The credit for this result, so important to our country and to civilization, belongs, in a large measure, to American civil engineers who went at once to the assistance of the Government.

It was the writer's privilege to visit the sanitary utilities at many of the cantonments, and he believes that, making due allowances for the uncertain factors which the problems presented to the engineers who designed them, they were remarkably successful. There was, however, at least one mischance to regret. Many of these structures were not large enough for the service which became necessary soon after they were installed. The rapidity and volume of mobilization in the camps was not fully anticipated, and the factors of safety in the capacity of the sanitary structures were small. Water supply, sewage disposal, and some of the other camp utilities, were not designed on a liberal scale, and it happened, sometimes, that where provision had been made to serve not more than 25 000 men, a service for 50 000 was necessary within a few weeks of the date of completion.

The writer in his various visits did not at any time see these utilities of too large a size, and, frequently, they were not large enough to meet the requirements. It was so easy to send men to a camp by the

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trainload and make extra accommodations, if necessary, for them, and so difficult to extend the water supply and the sewage disposal plants, that it was not surprising to find the latter overloaded and put out of service by this unexpected burden. Under these conditions, the quantity of sewage probably did not increase as much as might have been expected, in proportion to the enlarged population, or as much as the quantity of suspensa which it carried, and especially the quantity of fats.

The sewage being treated by the disposal plants was the strongest the writer has ever seen, and it was very fresh, causing the tanks to clog rapidly with scum and fail to function as treatment units, unless the scum was removed frequently, such removal in some cases being required daily.

It was unfortunate, in view of this experience, that a larger provision was not made in the design of these utilities for increases in camp population beyond the first estimates. In all the sanitary structures, a liberal policy in expenditure would have been justified, as the health and comfort of the men in training was of the greatest moment, and the extra cost required to provide for contingencies would have been only a small proportion of the total cost of the camps.

In providing sewage disposal plants for the housing projects of the United States Shipping Board, Emergency Fleet Corporation, account was taken of the probability, or at least possibility, that the villages would be occupied long after the end of their emergency use; that they would afford permanent homes for families; and, in many cases, would be absorbed into neighboring cities and towns. The utilities, therefore, were put in with this in mind, but with all possible economy. Wherever the sewage could be led to existing plants, this was done. If conditions were proper, the disposal of sewage was by dilution; but there were several places where tank treatment was absolutely required, and the choice lay between septic tanks, Imhoff tanks, and Doten tanks. For these villages, the septic tank was considered improper, and, after much discussion, the Imhoff tank was rejected as not advisable. Granting their well-known excellent points, it was decided that Imhoff tanks should not be erected if the plant was to be within smelling distance of the dwellings, especially when treating such fresh sewage as would be the case at these villages.

Under the most favorable conditions, the Imhoff tank is the most costly of all types to build, as well as the most costly and difficult to operate successfully. It requires from six months to a year to ripen thoroughly and is never free from the danger of "frothing" and creating a nuisance. When these tanks give trouble, they are worse than useless, and that they frequently do give trouble, periodical literature abundantly shows. For very fresh sewage with much grease, the writer considers them inadvisable. Small installations are especially likely to give trouble. In all cases, the plants at these villages would have

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to be near houses occupied for residence purposes. Nevertheless, in one instance, an Imhoff tank already built was used, and did not make its kind any more popular by its performance. In another instance, a city of considerable size joined with the industrial city in building a battery of Imhoff tanks, the writer, at the request of Admiral Bowles, acting for the Shipping Board, with the consulting engineer employed by the city and for the project, in selecting the design. This plant is not yet completed. It was located, however, so that complaints are not anticipated on account of odors; should odors arise—and the plant is large enough to expect that—they will be given proper care.

The "Doten" tank had been adopted for the military cantonments, and many such tanks were already in service. One was erected at Camp Meade, where it was operated under the Constructing Quartermaster, who, at the time, happened to be Maj. E. B. Whitman, M. Am. Soc. C. E., a former associate of the writer on the Baltimore Sewage Commission. The writer visited Maj. Whitman and was able to make a study of the tanks, which, designed for about 25 000 men, had clogged and failed to function under the load of about 50 000. One learns most about a tank by seeing it act badly; we all learn by such failures. What the writer learned by observing the Camp Meade tanks in distress, convinced him that the type had some good points; at least, it would not froth and boil over like the Imhoff tank under similar conditions, and it did not smell unless disturbed.

It was after several visits to Camp Meade that the writer became acquainted with the author, who kindly gave the officials of the Shipping Board many data concerning the design and characteristics of the Doten tank.

The first impression of this tank was certainly discouraging; but a long experience in sewage experimentation suggested the idea that it was doing better than should be expected under the conditions as seen. Arrangements were effected whereby the writer was enabled to visit and study a number of these tanks in service at the various cantonment camps, among which were the following: Fort Myer, Camp Meade, Camp Holabird, Camp Dix, Camp Mills, Camp Merritt, Camp Upton, and several others.

The first visits made to these camps were in June and July, 1918, at a time when, in every instance, from two to three times the load designed for was being carried by the tanks, which were generally being blamed for failure to function by those who did not stop to reflect that any tank has a limit of capacity, beyond which it will fail. The writer felt convinced that, if treated fairly, they would not be failures. Indeed, he was much surprised with the manner in which these tanks carried the observed overload. The sewage was of extraordinary strength, coming from men only, highly fed, with a large proportion of meats. The sewage was not only very strong, but it carried a heavy

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grease content and a vast quantity of fresh scum-forming materials. Under the conditions observed, these tanks filled with scum, cemented together with grease, which could only be got rid of by removal from the top. The sludge was removed without trouble from the hopper bottoms by the sludge pipes, and was dried with only slight, if any, odors on the drying beds. The heavy scum, however, which would not sink, of course, did not get down deep enough to reach the sludge pipes.

The scum trouble was caused mainly by the grease; when this formed about the floating solids, they rose to the top. The mass of scum became a solid cake, which, on being removed, required burial in trenches in order to prevent a nuisance. To meet this difficulty, Maj. Doten introduced the grease trap described in his paper.

After observing the cantonment camp sewage and other conditions, the writer came to the conclusion that an Imhoff tank would have clogged and failed sooner under this excessive and abnormal service, and that, of the two types, the Imhoff tank probably would have caused a greater nuisance and would have been the more difficult to clean, as it would have become filled with fresh settled matter, below and above, and covered with fats long before digestion could have been established. It would never have had a fair chance to function properly. Indeed, in most cases, before a two-story tank could have been built and given time to ripen, so as to work normally, it would have been useless, as the war would have been over.

In the housing projects of the Shipping Board much fresh sewage, not unlike the cantonment sewage, was to be expected from restaurants, etc.; and it was believed that the Doten tank would give as good, if not better, service than the Imhoff tank.

It is probable that a self-cleansing fine screen, placed ahead of either of these tanks, would have made a great difference. With such a help, perhaps an Imhoff tank would have worked successfully, and the capacity of either tank might have been more than doubled. In some places, if not in most, a fine screen would have afforded sufficient treatment without the aid of tanks; but the grease problem would have given the apparatus much difficulty until it was overcome.

One of the most interesting items included in the paper is the tank or trap used by the author to overcome the grease difficulty. The grease carried by the sewage, as already stated, was remarkably great in volume. Numerous determinations of this include an average of 150 parts per million for all the plants.

The writer was informed of instances at various cantonments in which more than double this quantity had been noted. This is about ten times the usual quantity found in ordinary domestic sewage, and its effect was to fill up the disposal tanks with floating solids embedded in a grease cake. With this condition, no tank could function properly.

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Unlike ordinary floating solids, the grease was carried past the baffles and over weirs, so that it spread from the inlet to the outlet compartments of such plants as that at Camp Meade. Moreover, it passed through a cycle of changes which caused an acid condition in the tank, and bad odors when it was cleaned. Grease trouble was encountered at all the plants visited by the writer late in June and early in July, 1918, and he was enabled to see some of these plants in the process of being cleaned.

The grease traps adopted at first were formed of a chamber about 18 in. in diameter and 30 in. deep. As about four cans of very hot dish-water had to be discharged from each kitchen, after each meal, each as large as the "trap" it had to pass through, such a trap was useless. To meet this difficulty, the grease catcher or trap described by the author, was designed and built with good effect. After its construction, the work of the disposal plants was quite another story. Even though the tanks were heavily overloaded, the grease trouble practically disappeared, and, with it, to a great extent, the excessive scum formation. Moreover, the recovered grease was fairly clean, and sold for a good price. Many of these grease catchers were used on the writer's recommendation by the U. S. Shipping Board, not only in the industrial villages, where cafeterias and eating houses were provided with them, but, also, in connection with the restaurants and eating places in the larger shipyards.

No form of sewage treatment tank known to the writer would have operated successfully with this grease coming into it, least of all an Imhoff tank which would have had its flowing chamber clogged very quickly, and would have been in an acid condition throughout. It probably would have frothed and boiled over continually, causing active nuisance from sickening smells. It never would have produced "ripe" sludge. For sewage of the peculiar character and strength produced at the cantonments, after careful study of these plants, and by comparing them and the results obtained with other designs, the writer is convinced that the type selected was fortunate for the purpose. He agrees with the author that it is probably a fact that multiple-compartment tanks are more capable than any other form for protecting trickling filters and sand beds from grease carried in the sewage; but he also thinks that no form of tank will accomplish this object with entire success, unless an efficient grease catcher, or trap, is used as near as possible to the camp, or retaurant, kitchens, so as to take out the grease at the head of the line. By this method, 95% of grease can be recovered on the average, and sold at a profit; and if this is done, other forms of tanks might have a fair chance of success.

When operating within their capacity, under proper control, the author has shown that the Doten tanks effected a high degree of sedimentation. Even when grossly overloaded with sewage of unusual

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strength, the effluent was usually well clarified. The removal of more than 90% of settleable matter from a sewage carrying 464 parts per million of organic solids, is certainly an excellent result.

When these tanks were first designed, there were probably few data on which to determine their size, even for ordinary municipal sewage, and it is very improbable that any one could have foreseen the character of the sewage they would be required to treat. The writer does not know of any digestion tank that will treat successfully the character of scum formed in these tanks, if saturated throughout with heavy grease which preserves the organic contents to a remarkable degree from digestion agencies. Such scum was, in its keeping qualities, not unlike the "pemmican" used by travelers and Arctic explorers. Solids would not settle while associated with this grease, so there was small chance either of digestion or sedimentation; even silt adhered to this floating mass.

The question may well be asked, To what extent may tanks of this form be expected to digest sludge? This is not easy to answer. That digestion of sludge and solids takes place in them to a greater or less extent, and under very unfavorable conditions, was evident to the writer in his examination of them. Leonard Metcalf, M. Am. Soc. C. E., has been quoted in a recent publication as saying that of those engineers who passed on the design of these tanks originally, only Maj. Doten believed that they would act as digestion tanks. Mr. Metcalf attributes the difficulty in the functioning of these tanks "to the fact that they were operated as digestion tanks". He says that the other engineers felt that they should be regarded essentially as sedimentation tanks, and, if so operated, they would do the work of clarification, and that was all that was thought necessary in the majority of cases. As clarifiers of sewage, there is no doubt of their success. The writer, however, does not think that the failures to function, which he saw, were due to their operation as digestion tanks. He has formed the conclusion that the cause of the trouble was overloading as sedimentation tanks. Up to the time when trouble occurred, the tanks operated in their own way without intelligent care, neither as sedimentation nor as digestion tanks, without regard to their capacity for either purpose, and without the removal of the scum until they were clogged. With the great volume of sewage pouring into them, carrying from 150 to 200 parts per million of grease; with 464 parts per million of fresh, settleable, suspended, organic solids entering at a rate of double and even triple their designed capacity, they were only about able to act as grit and scum catchers. Under these circumstances, there was far more flotation of suspended solids than sedimentation. They simply filled with grease and scum from end to end, as at Camp Meade, having merely a "convoluted passage" through them when thus filled, through which the sewage passed at a high velocity to the outlet. The

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fact is, a tank designed for 25 000 men will not serve 50 000 men with success, even as a settling tank. No digestion was to be expected from tanks operated in this manner, and yet some did occur. In the Fort Myer plant, when it was working within its capacity, the digestion seemed to be very good, and a fairly ripe sludge was produced, which dried quickly on a bed, without odors. Since the plants at the various camps have been relieved of their great overload, and have been operated within their capacity, the writer is informed that good digestive activity is being shown at many places, and ripe sludge comparable with the best Imhoff sludge is being produced. This is the best answer to the question as to digestion. It would be interesting to give this, as well as the other features presented by these tanks, a more thorough experimental study. They have turned out in service much better than the writer would have expected, and show some advantages over other forms of tanks which, at least, makes them very interesting.

The Housing Department of the U. S. Shipping Board adopted a design of plant using these tanks, in which multiple units of full size were placed parallel with each other, the purpose being to keep one unit at a time in service on the daily sewage flow, until it had collected enough sludge and scum to interfere with its function as a sedimentation tank. It was then to be put out of active service and given a rest period, long enough to obtain complete digestion of the contents, when it would be discharged, the ripe sludge being dried on beds. Meanwhile, another unit would be in active service. In one plant, four such units were provided in a battery, and placed under the cover of one building. As a rule, at least two units were provided. How successfully this plan may work out, cannot be foretold, as the service has not been long enough in either of the plants to prove anything. So far, they are working very well. The first one to be put into its rest and digestion period has only recently been taken out of active service.

It has seemed to the writer that these tanks, which were designed for sedimentation and clarification of sewage, were not so well fitted for the digestion of sludge, and could not be depended on for this purpose. His reason for this is perhaps rather theoretical and derived from studies, made at the Brooklyn Experiment Station, on Imhoff tanks and separate digestion tanks, in which the results seemed to indicate that a complete separation of the sludge under digestion, from the flowing sewage, was desirable; and a minimum capacity, of 2 cu. ft. per capita below the slot lines, was necessary in the digestion chamber. This applies to Imhoff tanks treating domestic sewage, such tanks being not less than 24 ft. deep from the water line to the sludge outlet.

In the other tanks, however, conditions differ from the Imhoff tank. The series of compartments, baffles, weirs, etc., have a selective action

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on the settleable solids, and the finer the organic particles, the quicker may digestion be accomplished. There is no such assortment of sizes in the Imhoff tank, in which the sludge is not ripe until the most resistant of the larger solids in it ripen. It may be true, therefore, that Imhoff tank digestion requires more time and more space than would be the case in the middle and outlet compartments of the Doten tank.

On the other hand, it seems doubtful whether the scum and settled matters in the first compartment of the Doten tank will digest in any reasonable time, because of their size and resistance to bacterial action, even when the tank is out of service and at rest. No digestive action is to be expected in this compartment while it is in active service; but if the grease in the scum does not prevent such action, hydrolysis and solution take place, and the dissolved materials are carried over with the flow of sewage to the next compartment, where it appears that digestion occurs rapidly.

Thus, it seems possible that if a tank of this type can be designed so that the first compartment is made of just the proper size for the flotation and hydrolysis of scum, which will not accumulate more rapidly than it can be dissolved and hydrolized, with the second and third compartments designed so as to afford a proper capacity for digestion, it might prove a very important acquisition to the sanitary engineer. The outlet compartment appears to have a marked effect in reducing the finely divided and dispersed organic colloids, giving a high degree of clarification. This is especially marked where there are more than three compartments in the unit.

Among the Doten tanks in operation examined by the writer, possibly the most successful was that at Fort Myer, near Washington, D. C. This tank was working at about its designed capacity and was producing a good and well-clarified effluent and an excellent quality of sludge, which was odorless and dried readily on a sludge bed. The effluent was discharged through a dosing chamber on a trickling filter bed of the character and design referred to by the author, as follows:

"The filter has an undulating floor, the distributor pipes being placed on the summits and pipe-covered gutters in the depressions. Thus, the expensive system of under-drains, which is usually an important feature in plants for municipalities, was eliminated."

The medium depth averaged 5 ft., and the size of stone from ¾ in. to 2½ in. The effluent was stable after 20 days, which is certainly a good result. Although the result of this design was good, and the cost low, the writer's experience and observation rather lead him to favor the standard design of placing the under-drains for all but very small plants like this one, at a minimum depth of 6 ft. for the medium. The absence of sufficient under-drains might cause clogging of the bed with humus during an "unloading period" of the filter bed. The design

adopted by Maj. Doten, however, has certainly given some good results, and is worthy of further study. If the cost of filter beds can be kept down by this method of under-draining, more filters will be constructed. Mr. Hammond.

The time occupied in the closing cycle, in applying sewage to trickling filters, has been given much study by engineers. Continuous discharge on beds through fixed nozzles has not led to the best results. Plants using this method have never equalled the capacity per acre which has been secured by those designed for interrupted dosing, as obtained by intermittent applications, with dosing tank and siphon discharge, through sprinklers.

Experiments at Brooklyn gave the best results when the interval between doses was very short, but actual, and the dosing period was also short. A small siphon chamber proved to be the best. For this reason, the Brooklyn dosing chambers were reduced about 25% from the size first provided, thus increasing materially the permissible rate of sewage application per acre.

The limit of 2 000 000 gal. per acre, which the author thinks should not be exceeded, the writer believes to be low, and he would permit from 2 500 000 to 3 000 000 gal., with effective dosings of domestic sewage, on a bed 6 ft. deep. This figure depends greatly on the character of the sewage. He does not quite agree with the statement, as of general application to all conditions, that:

"The humus discharged from properly functioning trickling filters should not create an objectionable condition due to putrefaction. The putrescible character of the effluents from some deep trickling filters, and also the humus held in suspension, is due either to an inferior quality of tank effluent or to improper dosing."

The writer believes that the humus from all trickling filters is at times more or less putrescible. He has never seen an exception to this rule, and the reason for it, he thinks, has little to do with the tank or the dosing, unless this is very uneven or grossly improper. The main cause of it is the lower forms of life in the filter bed, such as worms, etc., which are frequently being discharged with the effluent and make up much of the so-called "humus"; the gelatinous casts that come away from the filter, which are often loosened by worms, are also putrescible, and when "unloading" takes place, as it usually does in the spring and summer, putrescibility of the suspensa in the effluent is one of its prominent features. Of course, it is admitted that improper dosing may possibly carry a stream of untreated sewage through the bed, or may cause unloading; also that highly septic sewage may affect the aerobic bacteria so as to start an unload; but, although these things are possible, they are infrequent. There is no doubt, however, that some of the best trickling filters in the world discharge a putrescible humus with their effluent. For this reason, secondary set-

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tling tanks are necessary, except in small plants, or where the discharge goes into a large body of water.

The writer thinks that the plan of Messrs. Metcalf and Eddy, introduced at Fitchburg, Mass., of pumping filter humus back to the Imhoff tanks, is an excellent provision, as he knows of no way in which this humus sludge can be dried on a bed without causing bad odors.

Mr. Sherman.

A. L. SHERMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—It has been the writer's privilege, during the past eighteen months, to be associated with the author in the design of the various structures connected with the sewage treatment plants at the different army camps and permanent posts throughout the United States, and the experience gained in overcoming the many difficulties encountered in this work has been very valuable. That the problem of successfully treating the concentrated, heavily grease-laden, army sewage has been practically solved is now generally admitted by those in touch with recent developments.

The early difficulties in the operation of the type of sewage tanks adopted for use in the army cantonments have been assumed by many to be functions of the tank itself, rather than an inherent part of the problem of treatment. The problem of treating a sewage containing 450 parts per million of suspended organic matter, including 150 parts per million of grease, besides an indeterminate quantity of garbage, was one which was new to sanitary engineers. The low unit capacity, 10 gal. per capita, used for the tanks first constructed, was selected with the realization that the great saving in first cost would be in part offset by increased cost of operation. It was also assumed that the maximum population, on which the 10-gal. unit capacity was based, would be reached only at intervals, and that the average would be much below this maximum. Lack of shipping and subsequent increases in the army programme resulted in the camps remaining at full strength during the entire winter and spring of 1917-18.

The removal of 70% of the suspended organic matter by sedimentation would result in the deposition in the tanks of 4.1 cu. ft., or 33 gal. per annum, of sludge containing 85% of moisture. This volume would be reduced perhaps 30% by digestion, or to 23 gal. per capita per annum. The grease alone, if entirely retained in the tanks, would amount to 19 lb. per capita per annum, or, on a 50% moisture basis, 38 lb., or 4.5 gal. of sludge due to grease. It should also be noted that the tanks were placed in use just prior to the winter season of 1917-18; they were immediately loaded above even their rated capacity, without any opportunity for a gradual "ripening", and were practically without attention for the ensuing six months. To the writer, the remarkable feature of the performance of these tanks is that they functioned at all.

* Mansfield, Mass.

Mr. Sherman.

As a contrast to the above, the results obtained during 1919, with competent operation, grease traps of the type described by the author installed throughout the camps, some minor changes in the design of the sewage tanks, and, in some cases, increases in the capacity per capita, furnish a more reliable index of the true value of this type of plant for sewage treatment.

The plant at Camp Merritt, N. J., which was used during 1919 as a debarkation camp for a large percentage of the troops arriving at New York City, is the most available one for inspection by members of the Society in and near New York City, and is a fair example of this type. The operation of the water supply and sewerage systems at Camp Merritt have been under the charge of Capt. Paul Molitor, formerly in charge of the operation of the sewage treatment plant at Chatham, N. J. Capt. Molitor has had in direct charge of the sewage treatment works, Mr. W. R. Teats, formerly of the Engineering Staff of the Pennsylvania State Board of Health, and they have taken great pride in the sewage treatment plant, which, contrary to the custom at many municipal plants, has been made attractive to the eye by grading, grassing, and the construction of gravel walks.

Capt. Molitor and Mr. Teats state that camp sewage is much more difficult to treat than municipal sewage, and that, on account of the large quantity of scum-forming material present, it would be practically impossible to handle the sewage in a two-story tank. This plant, during the spring of 1919, cared for the sewage of an average population of 26 000, with a gross per capita capacity of 28 gal., and a 95% removal of settleable solids.

The following extract from a recent report by Capt. Molitor is interesting, in view of various opinions expressed as to the ability of this type of plant to produce satisfactory digestion of sludge:

"During the month, 405 cu. yd. of liquid sludge was drawn from tanks to sludge drying beds. All of this sludge was of excellent quality, dark in color, practically odorless; in fact, no odors could be detected from it more than 50 ft. from the sludge drying beds. Weather conditions being favorable this sludge dried to forkable condition in from 5 to 7 days. The sedimentation tests show a high percentage of removal of suspended soils by the tanks, proving that if these tanks are operated with reasonable care and intelligence they will perform the work for which they are intended. No complaints have been received as to objectionable odors emanating from the plant. The plant continues to operate in a normal and satisfactory manner."

Mr. Brown.

J. F. Brown,* Assoc. M. Am. Soc. C. E. (by letter).—The type of sewage treatment plant adopted by the Construction Division of the Army for installation in the Army camps and industrial plants during the war, is so radically different from that ordinarily encountered in municipal practice that some discussion of the principles underlying its design and operation seems to be warranted.

* Kansas City, Mo.

Mr. Brown.

Maj. Doten has described in a general way the outstanding features of the standard sewage tank and the sections of the tank shown on Plate V give the details of the structure. It may be said that some of the details, particularly the rectangular sumps at the bottoms of the hoppers, are not typical of the standard design and are not found in any of the tanks serving the larger camps. The sloping baffle at the inlet end of the tank is another feature not typical of the earlier designs, having been adopted as a standard detail only after the recognition, on the part of the designing engineers of the Construction Division, of the presence of large quantities of grease in the sewage and the necessity of preventing the entrance of this material into the tanks. The original plans included vertical baffles at both the inlet and outlet ends, all compartments in any one tank were of equal dimensions, the slope of the floors of the hoppers was much flatter than that shown in Plate V, and there were no sumps under the lower ends of the sludge eduction columns.

In operation, sewage enters the tank at the water level and is immediately deflected downward by the inclined baffle. Particles of grease, and débris, such as match sticks, fruit skins, etc., which float on the incoming stream of sewage, are caught in front of the sloping baffle and may be skimmed off at frequent intervals, thus preventing the entrance of such materials into the tanks. The downward flowing sewage passes under the baffle and into the first compartment where, owing to the increase in area of the flow section and consequent decrease in the velocity of flow, the heavier particles of solid material are deposited. The lighter particles are carried on in the current under the vertical baffle at the outlet end of the first compartment, over the partition or weir wall and into the second compartment, where further sedimentation obtains. The same process is repeated in the third and fourth compartments, the effluent collected over the outlet weir into the effluent trough being practically free from suspended or settleable solids. It is the repetition of the sedimentation process, carried out in four separate stages, which accomplishes such a remarkable clarification of the sewage.

Many data are now available showing the results of extended tests of tanks in several of the larger camps, indicating the high efficiency of this type as a sedimentation tank, and it is hoped that Maj. Doten will publish them before his work in the Construction Division is concluded.

Coincident with the multi-stage sedimentation process is the multi-stage sludge digestion process which, unless the liquor in the tank has been "soured" by the products of decomposition of grease, has proceeded to a remarkable extent in many of the Construction Division sewage tanks. With the decomposition of organic solids and the consequent

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evolution of gas, particles of organic material which have been deposited in the inlet compartment, are comminuted or split up by the pressure of the gas generated within them. The gas thus generated rises into the liquor above the sludge, carrying with it partly digested particles of sludge. The finer gas-borne particles rising into the liquor are caught in the current of sewage and carried forward, either to be redeposited at a point nearer the outlet end of the compartment than that from which they were lifted from the sludge, or carried over the partition wall into the second compartment. The larger gas-borne particles ascend into the liquor above the sludge and continue to rise until the attached charge of gas is liberated into the air, when they settle again to the sludge layer. This process of comminution and carrying forward of the partly digested solids continues until the larger solids have been broken up into particles sufficiently small to be carried by the current over the partition wall into the second compartment.

In the second and in each of the succeeding compartments the process of comminution, gas generation, and carrying forward of the more finely divided particles continues, the final effluent from the tank being practically free from settleable solids, and the sludge accumulating in the final compartment being well digested and easily dried on sludge filters.

The inclined baffle at the outlet end of the fourth compartment is designed to deflect backward toward the center of the compartment the gas-borne particles that rise from the sludge deposits near the end of the chamber, thus preventing their passing out with the effluent, even at times when the digestive action in this chamber is progressing vigorously.

The most striking points of difference between the municipal sedimentation or digestion tanks and the standard Construction Division tank include improved arrangements for sludge withdrawal, a separate sludge eduction pipe being provided in each of the compartments, so that sludge can be withdrawn from any compartment; by means of the sludge pipes the accumulation of sludge in any compartment can be transferred into any other compartment for further digestion; and by means of the water pipe connection into the sludge line, the sludge accumulation in any compartment can be thoroughly agitated. These features facilitate the operation of the tank, the most important feature from this standpoint being the arrangement of piping in such a way that sludge can be transferred from one chamber to another. Although the normal processes of comminution and carrying forward of sludge particles tend to effect a restoration of sludge-storing capacity in the inlet compartments of the tank, it sometimes happens that, in periods of excessive sewage flow, or at times when the digestive action in the inlet compartment is weak, the accumulation of sludge in this chamber

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becomes excessive, and it is necessary to relieve the condition by transferring the sludge to one or more of the other compartments.

Important differences exist between the secondary treatment units and the auxiliary structures of the Army sewage plants and those of a similar type found in municipal sewage plants. The dosing tanks for all plants constructed in 1918 were designed for a holding capacity equal to the discharging capacity of the filter nozzles for 1 min., thus providing a maximum length of dosing period not exceeding 1½ min., at maximum rate of sewage flow. Several of the dosing tanks constructed in 1917 were altered during 1918, their capacity being reduced to this basis, a remarkable improvement in the filter effluent following each such alteration. Sprinkling filters were designed during 1918 with an effective depth of 5 ft., and with the exception of a few small ones treating sewage from Army hospitals, no filters were constructed of a depth greater than 6 ft. The areas of the filters were designed on the basis of treating the sewage from 30 000 persons per acre of filter, which is approximately equivalent to a sewage flow per acre per day of 2 000 000 gal.

The sludge filters, or drying beds, were much shallower than most municipal structures of this type, the total average depth of the filtering material being 15 in., made up of 12 in. of crushed stone and 3 in. of sand. The filters for sludge drying were not provided with concrete floors, nor were the inner slopes of the embankments paved. The under-drains consisted of vitrified sewer pipe laid in the depressions of the subgrade, each under-drain serving an area approximately 10 ft. in width. In designing sludge filters, an area of ½ sq. ft. per capita of population served was allowed. This was found to be ample; in fact, in one case, the sludge from the sewage tanks was dried with no difficulty on a very much less area per capita than that stated.

Secondary settling tanks were provided in a very small number of plants. These were designed with a holding capacity of about ½ hour's flow of sewage. Where disinfection of sewage effluent by liquid chlorine was carried out, the secondary settling tank was provided with a gas-tight concrete slab cover, water-sealed traps on both inlet and effluent lines, and in this tank the chlorine was applied to the sewage.

From January 1st, 1918, until March, 1919, the writer visited in his official capacity nearly all the sewage treatment plants installed by the Construction Division, as well as many plants installed by other Government agencies, and several municipal plants. Gross as has been the neglect of municipal sewage treatment plants—and every sanitary engineer knows that the operation of most municipal plants has been characterized by wanton neglect—the Army sewage plants fared even worse. During the latter part of 1917 and the early part of 1918, no definite military organization had yet been formed which would have charge of, and direct the operation of, the various camp utilities such

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as the water supply and distribution systems, the sewerage and sewage treatment plants, etc. Not only was there no skilled attention given to these plants, but even the most obvious needs were neglected. Sewer manholes were used as receptacles for cast-off clothing, kitchen garbage, and all sorts of débris; screen chambers, where these had been installed, were allowed to clog and fill to overflowing; sewage tanks were permitted to fill with accumulations of sludge and scum, and no attempt was made to keep the nozzles on the sprinkling filters clear. Even the most casual inspection would have shown the serious need of immediate attention. The one or two plants, notably the one at Camp Merritt, N. J., which were operated by attendants who possessed some experience in the working of sewage plants, were able to handle the suddenly varying and often excessive loads which came on them without any serious difficulty.

Early in the summer of 1918, with the organization of the Maintenance and Repair Branch of the Construction Division, many sanitary engineers were taken into this branch of the service, and placed in charge of the operation of the camp utilities. This arrangement brought about a marked improvement in the methods of operation of sewage plants, and in the months immediately following its adoption many of these plants were quickly brought into a satisfactory operating condition.

The writer has read with interest the discussions on Maj. Doten's paper. Some of the opinions stated therein are apparently based on observations made during the spring and early summer of 1918, when nearly all Army sewage plants were clogged and filled with grease, and in many cases all digestive action in the tanks had ceased, due to the acidifying of the liquid resulting from decomposition of grease. Up to this time, as previously mentioned, no effort had been made on the part of the local camp authorities to protect the tanks from the excessive quantity of grease discharged into the sewers from the camp kitchens, and the enormous quantities of débris and garbage which were wantonly deposited in the sewer lines. Under the better conditions resulting from the installation of efficient grease traps and coarse bar screens, and intelligent operation, the conditions have almost completely disappeared.

The writer questions the statement that efficient tank treatment may be accomplished by the so-called two-story tank, especially if such a tank were to be used for the treatment of fresh, concentrated, grease-laden camp sewage. Reports of the conditions of Imhoff tanks in several of the Army Aviation Fields covering the first half of 1918 do not indicate that any considerable success was attained in their operation. In fact, it was found that these tanks were absolutely inefficient both as sedimentation tanks and as digestion tanks.

Mr. Brown. While admitting that it is the consensus of opinion that the effluent from a septic tank cannot be treated by processes of bacterial oxidation —such as filtration through trickling filters—as advantageously as one which is not effected by the products of decomposition of sewage solids, the writer believes that the arguments advanced in support of this opinion are academic in their nature, and are not borne out by facts. He is of the opinion that the results of the series of chemical analyses made by the Bureau of Chemistry of the Department of Agriculture on both tank and filter effluents at the Fort Myer plant, which were quoted by Maj. Doten, indicate the fallacy of this opinion. It would appear, by logical deductions from the results of these tests, that the processes taking place in the Fort Myer tank rendered the effluent especially susceptible to treatment on a trickling filter. Maj. Doten has apparently reached a similar conclusion regarding the success attending the treatment of the effluent from a tank of the Construction Division type on a trickling filter. Additional evidence supporting this conclusion can undoubtedly be collected by a study of existing records of chemical analyses of such effluents, and it is hoped that Maj. Doten will present at least a summary of the data found in these records before the discussion of his paper is concluded.

Mr. Perry. Lynn E. Perry,* Assoc. M. Am. Soc. C. E. (by letter).—The first tank built by the Construction Division of the Quartermaster Corps which the writer had the good fortune to inspect was at Fort Myer, Va. This was in May and June, 1917. When his attention was first drawn to it, it was under construction, but was rapidly completed and placed in operation. The writer's opinion was that it would act as a clarification tank rather than as a sludge digestion tank. Its remarkable efficiency as a settling tank impressed him forcibly; especially as he remembers very distinctly, that the effluent was turned on the sprinkling filters when they were little more than one-third filled with stone. At that time, the effluent from the filters flowed through a very small ditch which showed no evidence of carrying an unstable flow during his stay at Fort Myer.

In discussing the design of this tank with H. C. McRae, Assoc. M. Am. Soc. C. E., at that time, the writer did not believe that the advantage of the pyramidal bottom was commensurate with its cost; it was not sufficiently steep to force the sludge down, although it was too steep for men to stand on while cleaning the tank.

After reading Maj. Doten's paper and learning that an attempt has been made to obtain efficient digestion in these tanks, the idea has occurred to the writer that probably better results might have been obtained if the tanks had been constructed so that the flow could be reversed, horizontally. This has been done in some recent designs of Imhoff tanks and although the writer has not observed any consider-

* Easton, Pa.

Mr. Perry.

able improvement, a logical attempt, at least, has been made to help the operator over his difficulties.

The writer cannot refrain from expressing his commendation to Maj. Doten for not adopting some form of the Imhoff tank as a standard of the Construction Division. Due to certain propaganda, almost all the laity and a large percentage of engineers regard the Imhoff tank as a panacea for all sewage ills. Indeed, it was stated by at least one sanitary engineer, in an address before a convention of Sewage Plant Superintendents, that in the design of the Imhoff tank a totally satisfactory method of treating sewage had been obtained. Those who have had to do with the operation of Imhoff tanks, mildly speaking, are skeptical of such an assertion.

Several series of investigations have been conducted in an endeavor to learn the cause of "foaming", or what measures will stop or prevent "foaming". These have been almost fruitless. One investigation seemed to indicate that when a tank was placed in operation in the autumn or winter the low temperature inhibited the growth of liquefying bacteria; the sludge became acid and resulted in "foaming". On the other hand plants placed in operation during the spring or early summer digested their sludge well before cold weather; the sludge was highly alkaline, sufficiently so to keep the digestive process going until warm weather again arrived. The number of plants considered in this investigation, however, was small, and mention of it is made merely to indicate how thoroughly the writer has gone into the operation of Imhoff tanks in an endeavor to secure uniformly satisfactory results.

There has never been sufficient co-operation between the designers and operators of sewage plants to obtain the best results. It would seem that the Army proved no exception. Having been interested from both angles, the writer will not comment further.

Early in the war, the writer was assigned to one of the Aviation Fields, where the garbage problem was handled by the duplicate can system. No garbage cans were ever taken into the kitchen; they were kept on a screen-covered fly-proof platform some distance from the kitchen door. The wagons came alongside the platform, loaded the full cans without removing the covers, and hauled them away (in this case 10 miles). The cans were emptied there, scalded and rinsed in boiling water, and returned the next day. The garbage was fed to hogs. The entire process was arranged for and inspected by the writer, who was Sanitary Inspector.

The paper was collected and delivered to the Quartermaster who looked after its baling and disposal. Although an incinerator was provided, it was not used.

Each kitchen was supplied with a grease-trap which was cleaned every morning, the grease being stored in barrels with fly-proof covers. The grease was afterward disposed of by the Quartermaster.

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The sewage treatment plant consisted of septic tanks, a contact bed, and final disinfection. The twin septic tanks were covered and had a capacity of about 8 hours' flow, but due to the fact that all the flow was during 16 hours of the day, actual storage was somewhat less than 6 hours when the plant was placed in operation. Scum accumulated rapidly and reduced this storage to half, resulting in a very inefficient arrangement.

The contact bed was dosed by a siphon, but the bed was very small and the dosing tank very large, which resulted in the bed becoming full before the dosing tank emptied; some of each dose, therefore, was only strained instead of aerated.

There was no final settling or chlorine contact tank; the effluent from the contact bed was dosed as it passed under the chlorinator house. The chlorinator was a Wallace and Tiernan Type "A" Manual Control machine which was designed for a uniform flow rather than a pulsating flow from a contact bed.

The population contributing to this plant was somewhat more than 1 000, within 10% of the estimated population. The daily per capita water consumption was approximately 140 gal., although four times that quantity was available and the sewage plant was certainly not designed for more than 40 gal. The sewage plant was poorly designed and indicated little or no knowledge of the water supply.

The writer apprehended operating difficulties when the plans were shown him by the Construction Engineer, and took the matter up with Washington immediately, but the design was never changed. Although the Construction Engineer had to build the plant in accordance with plans and instructions from Washington, he secured permission to use about 1 000 ft. of sewer pipe for an outfall line to carry the plant effluent far enough away to prevent a local nuisance. As this effluent flowed about five miles through an open ditch, with almost no dilution, before being discharged into the river, it is plain that some complaints would follow. There was no complaint, however, while the plant was under the writer's observation, but it is seen that operation difficulties were present in that:

1.—Insufficient sedimentation was obtained; the sludge was not digested and was hard to handle.

2.—The contact bed operated merely as a strainer (analysis indicated that it did no good whatever).

3.—The chlorinator and appurtenances were of the wrong type, resulting in inefficiency.

No relief was obtainable, and the operator was responsible for the quality of the effluent. This constitutes the writer's personal experience with army sewage plant operation in the United States. Although no complaints were filed in this case, the plant was far from efficient, a fault for which the operator was not responsible.

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The experiences of a single observer are far too narrow for general statements; a soldier sees only the details immediately before him, and never has an opportunity to obtain a knowledge of comprehensive plans or the difficulties under which the officer at Headquarters has to contend. If based entirely on personal experience and observation, the writer's opinion concerning the activities of the Construction Division would be entirely different from the fair estimate that has been made possible by Maj. Doten's paper. His experience was similar to hundreds of other officers and, no doubt, the information contained in this paper will throw a new light to some of the critics of the War Department.

Mr. Doten.

Leonard S. Doten,* M. Am. Soc. C. E. (by letter).—The writer in his paper endeavored to give a brief but comprehensive outline of the work in sanitation which had been accomplished by the Construction Division of the Army covering the period of the war to December, 1918. Special effort was made to present the facts as clearly and concisely as possible, all references to details of office organization and personnel, structural details of design, and other more or less extraneous matter having been purposely omitted. It is felt that too frequently the technical elements of papers on engineering subjects are subordinated to those savoring of commercialism, and as a result the factors of real value in the advancement of engineering practice are often obscured.

It is very gratifying that so many engineers have responded to the invitation to discuss this paper. In the writer's estimation these discussions in most cases have effectually supplemented the paper and are in themselves valuable contributions to the literature on sanitation.

After reviewing these discussions it is evident that most interest has been shown in the subject of sewage treatment. It had been anticipated that a more general discussion would be brought out on all topics, including collection, classification, and disposal of solid wastes, garbage can-washing equipment, and incinerators, as all these are important features in present-day sanitation work and will be of greater importance in the years to come.

As many requests, both verbal and written, have been received for data on results of operation of sewage treatment plants, and for information as to the best methods of operation, they will be complied with quite fully in this discussion. Furthermore, in order to correct a false impression which prevails to a certain extent, the writer takes this opportunity to state that neither the tank designs nor any other sanitary devices illustrated or described in the paper are patented.

The sewage tanks are so different in design from the European types, Cameron and Imhoff, which have been so commonly adopted for use in the United States, it is not strange that engineers unfamiliar with their development and having made only a superficial inspection

* Washington, D. C.

Mr. Doten.

of some of the earlier plants at the camps, should not readily appreciate the advantages of certain details of design.

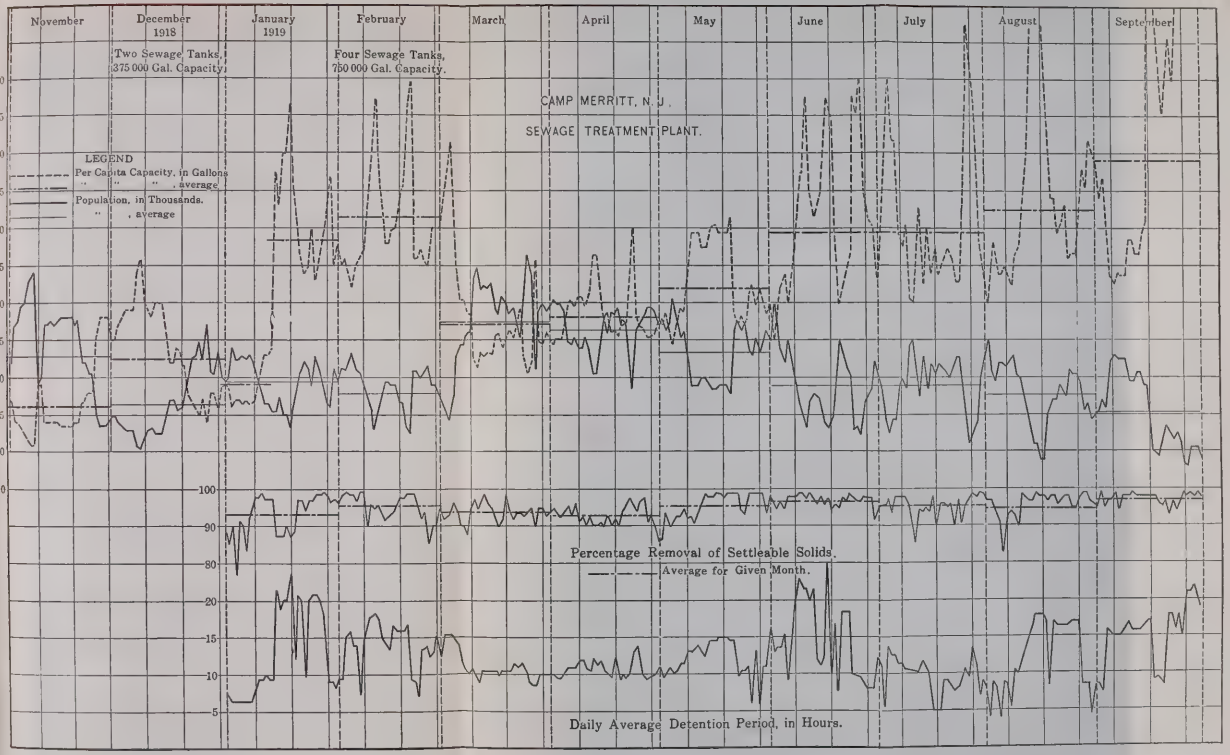
The adherents of the Imhoff principle were quite prone to use the term "sedimentation" in referring to the tanks, and were disinclined to acknowledge any evidence showing that they were not only "sedimentation" but also "digestion" tanks. Although opinion in regard to this matter has radically changed, yet it is probable that a few are still unwilling to admit the dual capacity of the tanks. It is evident from their discussions that Mr. Metcalf, Mr. Hill, and Mr. Fales are members of the first-named class. The writer's position on this question was clearly stated in his paper. It is a fact readily substantiated that to-day all the tanks are operated as digestion tanks and are functioning successfully.

Mr. Metcalf seems to have been misinformed as to the meaning of the instructions in the 1917 general specifications relative to requirements for capacities of tanks. The gross holding capacity was obtained by multiplying the number representing the maximum population of a camp, as determined by the rated capacity of the barracks and quarters, by 10 gal. The maximum population was used and not an average. It was assumed that the population would fluctuate widely. The large populations, in numerous instances greatly in excess of the rated housing capacity of the camps, and the continuous occupancy of the camps during the first eight to ten months, were the result of inadequate transport service due to lack of ships. During the latter part of 1918 this condition did not generally obtain, and the camp populations fluctuated much as had been anticipated in the early days of the war.

Mr. Hill's discussion of certain points relating to tank and grease-trap characteristics is so admirably frank that the writer feels there is a certain degree of impropriety in controverting his statements by using data obtained from the plant at Camp Merritt, where he was one of the supervising engineers during construction. The records of operation at this camp, however, are more complete than at the other camps, for which fact we are greatly indebted to Capt. Paul Molitor, the officer in charge of the plant, and, consequently, most valuable not only as an answer to Mr. Hill's criticisms, but to engineers interested in the study of sewage treatment processes. Although the results obtained at the Camp Merritt plant are not superior to those at many of the other plants, on account of the completeness and length of period over which the records extend, it was concluded to present the data in graphical form. Attention is invited to Plate VI. Furthermore, it may be of interest to Mr. Hill to learn that 221 756 lb. of grease in the aggregate was reclaimed at Camp Merritt from the kitchen traps. The following quotation is taken from one of the monthly reports:

"All grease traps were inspected four times during the month and scraped and the grease removed * * *, a total of 19 542 lb. being obtained, which was sold by the Salvage Department for $781.68."

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1432.
DOTEN ON
SEWAGE AND WASTES DISPOSAL.



Mr. Doten.

Mr. Doten.

By a careful inspection of the chart, Plate VI, and from the quoted paragraphs from reports on operation, it will be noted that the following are significant facts: (*a*) uniformly high percentage of removal of settleable solids, irrespective of season and rate of sewage flow; (*b*) rapid fluctuation in rate of flow and population; (*c*) digestion in tanks going on satisfactorily throughout the whole period; (*d*) very little odor about the plant; (*e*) effluent at all times in readily oxidizable condition; (*f*) all tanks were operating in parallel. The effluent was treated with liquid chlorine and discharged into a small tidal stream without creating any nuisance. In this connection, attention is called to the quotation from Capt. Molitor's report for September, 1919, which is given on page 381 of Mr. Sherman's discussion.

The following data are from reports on operation:

Camp Eustis, Va., August, 1919.—"Average daily sewage flow, 732 580 gal." * * * "All units of sewage disposal plant are in operation * * *. Sewage tank operation for month of August was satisfactory. 1 429 cu. ft. of sludge were removed on August 18th, from bays No. 2-A and No. 1-A. Sludge was dried out in a period of five days and carted away to farm."

From other reports received it has been learned that no scum forms on the sewage in these tanks, and that thoroughly digested sludge was produced in about six weeks from the time they were first put into use.

Camp Eustis, Va., September, 1919.—The following statement is contained in the monthly report prepared by the Sanitary Engineer at the camp and submitted to the Surgeon General of the Army: "The sludge was well digested and contained about 87% water." Similar reports have been received in monthly reports since that date.

Camp Merritt, N. J., July, 1919.—"Average daily sewage flow, 1 789 000 gal." * * * "The four tanks have been in operation during the month and are functioning in a normal and satisfactory manner, both as to clarification and sludge digestion. The sludge drawn from these tanks during the month was of good quality, well digested, very dark in color, and practically odorless, with the exception of a slight tar or burnt rubber odor, which is characteristic of thoroughly digested sewage sludge. The dry sludge is being carted away by farmers to be used as fertilizer. Weather conditions being favorable, this sludge would dry to forkable condition in from five to seven days."

Camp Sherman, Ohio, April, 1919.—"The new screens and grit chambers are operating satisfactorily and likewise the new sludge beds. The sludge is so thoroughly treated and dried that it is practically impossible to detect any odor whatever in the residual."

Fort Myer, Va., September, 1918.—The report of Dr. W. W. Skinner and Mr. J. W. Sale states:

"The sludge that is removed from the septic tank is disposed of by burying. It is not offensive, when dry, having only a slightly musty odor. The appearance and odor of this material, therefore, could not indicate its origin to the uninitiated."

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Camp Upton, N. Y., January, 1920.—The following endorsement was received from the Utilities Officer at this camp:

"Returned, with the information that we have experienced no difficulty in the operation of the sewage plant, except the normal one of developing new tanks and new sprinkling filter to the point of maximum efficiency, and that through our experience of the past seven months we have arrived at practically the same methods as are described in recent 'Instructions Relative to the Operation and Maintenance of Sewage Treatment Plants', dated at Washington, December 1st, 1919. We have brought our practice into line with these instructions where they differed in any details. * * *"

The stability of effluent from the trickling filter for the preceding month was given as 99 per cent.

The significance of the preceding statements is apparent to those engaged in the design of the plants. A misconception on the part of the earlier plant operators and their advisers as to the fundamental principles involved in the processes caused great delay in establishing effective and efficient methods. Since the stress of war conditions has passed somewhat, the writer has succeeded in bringing about more uniform methods of operation. A complete revision of the rules for the operation of sewage treatment plants was made in August, 1919. The following quoted paragraphs are taken from the revised operating instructions:

"*4.—Sewage Tanks.*—It is desirable to maintain an equal rate of flow through all tanks in service. To accomplish this, in certain cases it may be necessary to adjust the valves on inlet pipe lines or gates in distribution chambers. A uniform depth of flow over the effluent weirs of all tanks in service is an indication of proper distribution of flow in the several tanks.

"*5.—Number of Tanks to be Used.*—Where two or more tanks are provided, the operator shall use at one time such number of tanks as will provide an average detention period of not less than six hours nor more than twelve hours. In computing the detention period the entire capacity of each tank should be used.

* * * * * * *

"*8.*—During the first few months of operation it is essential to break up the scum daily by means of a hose stream or by aid of some mechanical device in order to avoid the formation of a heavy scum.

"When the bacterial development in the tank, especially the first compartment, has reached a certain stage, hosing is required less frequently.

"Scum formation varies considerably in different tanks, being dependent upon certain characteristics of the sewage. The important feature, not to be neglected in any particular, is the necessity of preventing the formation of a dense thick mass of scum.

"*9.—Lime Treatment for Acid Sewage.*—Wherever the litmus test shows an acid reaction in the raw sewage, milk of lime should be added in quantity sufficient to neutralize this acidity. The quantity of lime

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required will vary from 100 lb. to 500 lb. per million gallons, depending upon the degree of acidity of the sewage.

"*10.—Transfer of Sludge.*—At times it may become necessary to transfer partially digested sludge from the first compartment of a tank to succeeding compartments, in order to facilitate digestion. This is especially important where tanks are being used to their maximum capacity. The transfer may be accomplished either by means of a gasoline-operated diaphragm pump, or by drawing down the liquid in the last compartments through the draw-down valves and then transferring sludge through the sludge lines.

"Where the sludge pipe system is provided with a direct connection with a water distributing system it is desirable to agitate the sludge in the bottom of the first two compartments by the introduction of water through the sludge lines. This should be done in five-minute periods, weekly.

"*11.—Drawing of Sludge.*—Sludge should be drawn only when thoroughly digested. Sludge in the last compartment should be removed frequently in order to obtain an effluent low in turbidity.

"Sludge shall be drawn into sludge beds to a depth of not more than 12 in., after which sludge lines shall be drained or filled with water, or clarified sewage, to avoid formation of heavy deposits. Digested sludge should dry to a spadeable condition in 3 to 6 days, according to weather conditions. When sufficiently dried, the sludge should be removed from the beds and deposited in fills or disposed of as fertilizer.

* * * * * * *

"*Operators' Records.—19.*—Daily records should be kept of the amount of sewage treated; the maximum and minimum flow; the temperature of the sewage; the amount of rainfall; the quantity of screenings and detritus removed; quantity of disinfectant used; the quantity of sludge drawn on to the beds and dried; the results of the tests above outlined; and of the methods and cost of rectifying any unusual conditions arising in the operation of the plant."

In regard to Mr. Fales' reference to three standards of efficiency for sewage tanks, the writer desires to state that all three requirements have been fully met by the tanks under consideration, as shown by results of operation during the past two years. Mr. Fales' observations were largely made previous to the installation of grease-recovery tanks. Consequently, he did not have the advantage of observing the satisfactory operating conditions which followed. In this connection, attention is called to the fact that the original sewage treatment plant at Camp Dix, N. J., was constructed in accordance with plans prepared by the firm of which Mr. Fales is a member. It differed in several important details from the standard plans of the Construction Division. During the latter part of 1918 this plant was considerably enlarged by the addition of tanks and filters of the 1918 standard Construction Division types. This, with one other exception, is the only plant that was enlarged to the extent referred to by Mr. Fales as having been recommended by the Consulting Engineers. In both places trouble was

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experienced in operation on account of the acidity of the sewage resulting from low alkalinity in the water. This condition has since been overcome and the plants are now in exceptionally fine operating condition. Grease traps installed at the kitchens at Camp Dix have intercepted 362 000 lb. of grease, which has been sold for $18 000. More than 180 tons of grease therefore was kept out of the sewage tanks.

The plant at Camp Dix, as also that at Fort Sheridan, presents an interesting object lesson on the effect of baffling in single-story tanks. The old tanks at the former plant were constructed of wood and contained simply a narrow scum board at the outlet end of each tank. The new tanks have the standard type of baffling and sludge dams, shown in Plate V, except that the baffles are constructed of wood instead of concrete. It was found to be impossible at times to maintain the trickling filter in serviceable condition when the old tanks were in service, due to grease and the carrying over of gross suspended particles. On the other hand, the new tanks with even shorter detention periods yield an excellent effluent. The nozzles in the old filter at Fort Sheridan became clogged in a very short time when the old hydrolytic tanks were temporarily placed in service, but the new tanks with about the same detention period remove from 96 to 100% of all settleable solids.

The Fort Sheridan plant has been in service one year, and has been heavily loaded continuously during that period. As further proof that there are no gross particles of suspended matter passing from the new tanks, it is found that the two fine-mesh screens located in the effluent flume, shown in Plate V, intercept negligible quantities of such matter. These screens have $\frac{3}{16}$-in. openings.

As Mr. Allen's discussion deals wholly with the type of tank which was adopted by another Government Department, no comment will be made except to state that the Great Lakes Naval Training Station plant was not functioning satisfactorily at the time the writer inspected it in July, 1918, the aerating flume and rapid sand filter then being out of service. It is much regretted that Mr. Allen did not see fit to discuss the type of plant referred to in the paper, but it is assumed that his reluctance to do so was the result of lack of experience with the camp plant installations.

Mr. Kershaw's discussion is very interesting. The comparison of the test data on British camp sewage presented by him with the results of tests obtained in camps in this country, shows that the sewage in the latter camps was much more concentrated and that the quantity of suspended matter was much greater. The extremely high organic suspended matter content of the camp sewages was one of the surprising factors of the sewage treatment problem.

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Judging from the description of the typical sewage treatment plant adopted by the British Government, it is concluded that the cost per capita for such plants, if they had been constructed in this country, would have greatly exceeded the cost of the plants which were built by the Construction Division.

When it was learned that the Housing Department of the United States Shipping Board had adopted the design of plant described in the paper as its standard, the writer considered it a distinct compliment to the Construction Division, and especially later when it became known that this decision had been reached as the result of Mr. Hammond's recommendation after making a thorough inspection of a number of the War Department sewage treatment plants. It was reassuring to find that an engineer of such broad experience, who had gone deeply into the problem, had reached conclusions very similar to those arrived at by the writer. Mr. Hammond's discussion merits careful consideration by all engineers interested in this subject and should prove of great value in the advancement of engineering practice in sewage treatment.

Mr. Besselievre has stated clearly the advantages resulting from close supervision of sewage treatment plants by the designing engineer. It is so obviously the correct method to adopt that it does not seem to the writer to be a debatable question.

It is usually possible to improve the grounds about a plant at small cost so as to make them attractive landscape features. This should be done wherever practicable, as it stimulates pride in the plant on the part of the plant operator and municipal authorities and thus insures proper attention to details of operation and, consequently, satisfactory results unless the plant design is grossly at fault.

In order to secure the best results with trickling filters, designers should be careful to observe the points relative to details of design of dosing tanks and distributing pipe systems brought out in Mr. Besselievre's discussion. The advantages of such designs may be seen at many of the camp plants. The new filters at Fort Sheridan, Ill., and Camp Dix, N. J., are good illustrations.

The writer unintentionally omitted in his paper to state that the design of tank shown in Plate V was not typical of the earlier designs. Capt. Brown's reference to that fact, therefore, is appreciated. The design illustrated in Plate V is one of the last designs prepared in the Construction Division previous to the presentation of the paper, and was included among the other illustrations to show the latest practice. The design referred to by Capt. Brown as typical of the earlier practice is illustrated in *Engineering News-Record*, of November 15th, 1917.

The inclined baffle at the outlet end of the tanks was used early in 1918, but the inclined baffle at the inlet end was not used until the latter part of the same year. Both features, together with the sump

Mr. Doten. in the bottom of each compartment, are included in the present standard plans, as will be seen by reference to Plate VII.

Capt. Brown's intimate knowledge of the details of design and principles of operation is evidenced by the excellent description which he has given of the action which takes place in the several compartments of the multiple-compartment sewage tanks. He also clearly describes the conditions at some of the camps previous to the installation of effective grease traps and to the organization of the operating forces.

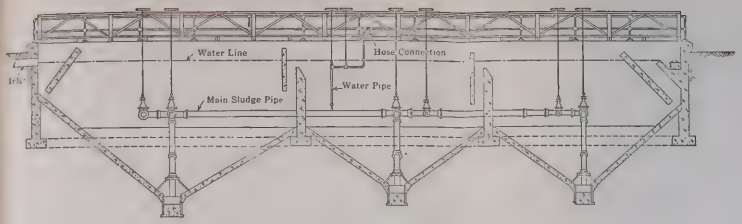
Mr. Perry's conclusion, as well as that of Mr. Metcalf, relative to the difficulties which might be anticipated in the withdrawal of sludge from the compartments with pyramidal bottoms, is not confirmed by experience in the operation of the tanks. The sludge is readily drawn from the tanks without interrupting their use; it simply requires the opening of a valve. Rarely is it necessary to de-water a compartment to remove sludge. The writer has not learned of an instance of this kind during the past 1½ years, or since the early days of camp occupation, when due to misuse of the collecting system or to lack of proper attention to operation, the sludge withdrawal pipes became clogged with old clothing, large quantities of raw vegetables, or sand and gravel.

If designers in the past had devoted as much study to the design of tanks to meet the exact requirements of each individual problem as they have in their endeavor to fathom the mysteries of Imhoff tanks, their efforts would have met with much greater success. This statement is prompted by the expressions contained in the following quotation from Mr. Perry's interesting discussion:

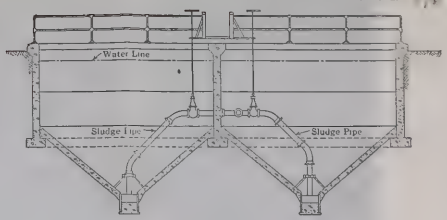
"The writer cannot refrain from expressing his commendation to Maj. Doten for not adopting some form of the Imhoff tank as a standard of the Construction Division. Due to certain propaganda, almost all the laity and a large percentage of engineers regard the Imhoff tank as a panacea for all sewage ills. Indeed, it was stated by at least one sanitary engineer, in an address to a convention of Sewage Plant Superintendents, that in the design of the Imhoff tank a totally satisfactory method of treating sewage had been obtained. Those who have had to do with the operation of Imhoff tanks, mildly speaking, are skeptical of such an assertion."

The last portion of Mr. Perry's discussion apparently relates to conditions at one of the aviation fields with which neither the Construction Division nor the writer had any official connection. It is regretted that Mr. Perry did not bring out this fact more clearly.

Experimental Work.—This paper would be incomplete without at least a brief review of the experimental work and tests made at several of the sewage treatment plants. In April, 1918, Capt. E. J. Tucker and Lieut. J. J. Newman, Sanitary Corps Officers on the Staff of the Surgeon General of the Army, carried out a series of tests on the camp sewage treatment plant at Fort Myer, Va. In May of the same year



LONGITUDINAL SECTION



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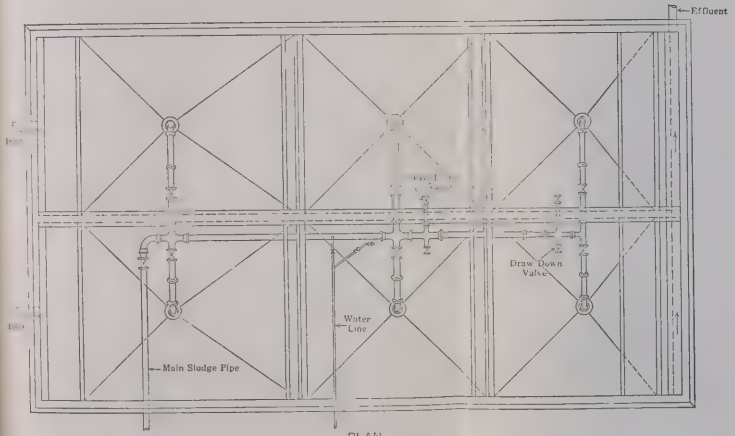

PLAN

MULTIPLE-STAGE SEWAGE TANKS

TYPE B

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Mr. Doten.

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similar tests were carried out at the Camp Meade plant. Each of these tests covered a period of seven days. Samples of the raw sewage, tank effluent, and filter effluent, were taken at the Fort Myer plant at 15-min. intervals. The work was carefully carried out, proper allowance being made for the variable rates of flow in weighting results of analyses. As a result of this investigation Capt. Tucker arrived at the following conclusions: Mr. Doten.

"(*a*) The sewage at the cantonments is strong and concentrated, more comparable to that produced in European cities than to our American cities.

"(*b*) The sewage is delivered to the treatment plant at widely fluctuating rates, but due to the routine activities of the camps the periods of maximum and minimum flow occur at practically the same time daily. The maximum rate is about twice the average rate and the minimum is less than one-fourth the average. Knowledge of the daily flow and the variations in flow is of the greatest importance since it determines the design of the sewage treatment plant.

"(*c*) The sewage contains an abnormal quantity of grease which should be recovered both on account of its economic value and because it hinders the bacterial activities so essential to the digestion of solids in the sewage treatment tank."

The average grease content as determined by the series of tests at Fort Myer was approximately 115 parts per million. The tank removed 71% of the total suspended matter, 72.5% of the volatile suspended solids, and 77.6% of the fixed suspended solids. No determinations were made with conical glasses to obtain the percentage of reduction of settleable solids. It was found that the crude sewage contained an average of 717 parts per million of volatile solids and 500 parts per million of volatile suspended solids. The conclusion was reached that "allowing for variations in flow, accumulation of solids, and digestion of solids, it would appear that 20 gal. per capita should be the least that should be considered in arriving at the size of the tanks for the various cantonments."

In June, 1918, following these tests, effective grease traps were installed at all the company kitchens in the camp at Fort Myer, Va. The effect of the removal of large quantities of grease from the kitchen waste water was very noticeable in the improvement which took place in the sewage treatment plant.

Following the installation of these traps, Capt. Tucker carried out an interesting series of experiments to determine the efficiency of the traps. The report is too voluminous to be incorporated in this discussion, but the following are some of the conclusions reached:

"Therefore a unit capacity of 0.5 gal. per capita is ample size for any grease trap.

"The attached curves show the efficiency of grease removal of a trap of 115 gal. capacity, with respect to rate of flow in gallons per hour

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and temperature of the effluent. The conclusions to be drawn from these curves are that: (*a*) Efficiency is not reduced appreciably with ordinary flows which vary from 50 to 150 gal. per hour, and there is considerable loss in efficiency when rates are above 150 gal. per hour—in other words, when the flow per hour is less than or slightly more than the capacity of the trap, a good grease removal can be expected; at rates per hour 100 to 200% higher than the capacity of the trap there is a marked reduction in efficiency; (*b*) temperature of effluent has but slight effect except when accompanied by high rates of flow."

From the results of these tests and others that were carried out under the direction of Mr. A. B. Dunning, of the Construction Division, at Fort Myer and Camp Meigs, the conclusion was drawn by the writer that a grease trap suitable for camp use should have a capacity of about $\frac{2}{3}$ gal. per capita; that it should be designed so that the settleable matter would not be retained in the trap; that it was very important to keep the velocities in the traps as low as possible; and that the cooling of the liquid was of less importance than the control of currents. These ideas, together with the large liquid surface area for the collection of the grease scum, were the most important factors entering into the design of the trap adopted by the Construction Division as a standard.

The examination of the sewage at Camp Meade, which was made by Lieut. Newman, in May, 1917, confirmed to a large degree the conclusions previously reached at Fort Myer relative to the camp sewage characteristics. It was found, however, that the grease content at Camp Meade was considerably higher than that at Fort Myer, averaging about 154 parts per million.

The experimental work carried out at Fort Myer and Camp Meade in April and May, 1917, by Capt. Tucker and Lieut. Newman, was of much value in determining the characteristics of camp sewage and in bringing about better operation of plants.

A series of tests to determine the characteristics of the raw sewage and the efficiency of the tanks in the removal of suspended matter was carried out at Camp Custer, Mich., by Capt. C. B. Hoover, of the Construction Division, and Capt. F. E. Daniels, of the Sanitary Corps, in the fall of 1918. The tests began on August 9th and terminated on November 15th, and were made before grease traps of the improved type were installed. From these tests it was found that the average total suspended matter in the raw sewage was 594 parts per million, and the average volatile suspended matter was 476 parts per million. These results were obtained from continuous records covering a period of 43 days. The water consumption during that period averaged 42 gal. per capita. The results of the tests checked closely with the earlier tests at Fort Myer.

In September, 1918, a chemical and bacteriological examination of tank and trickling filter effluents was carried out by Dr. W. W. Skinner

and Mr. J. W. Sale, of the Bureau of Chemistry, Department of Agriculture. This investigation covered the period from September 18th to the 24th, inclusive. The following are brief extracts from the interesting report submitted by them as a result of that investigation. More complete data are given in a paper recently prepared by them and read before the American Chemical Society, at St. Louis, Mo., on April 15th, 1920. Mr. Doten.

"Data from the Boston and Reading plants are given for comparison:

	Septic effluent, parts per million.		Average.	
Total organic nitrogen...	4.7	Boston, 1907...	3.1	Fort Myer
Free ammonia.....	18.7	" " ...	19.0	"
Nitrite	0.0	" " ...	0.0	"
Nitrate	0.0	" " ...	0.0	"
Oxygen required.....	26.0	Reading, 1909.	32.0	"

	Trickling filter effluent, parts per million.		Average.		
Total organic nitrogen...	3.4	Boston, 1907...	0.7	Fort Myer,	1918
Free ammonia.....	12.0	" " ...	1.8	"	"
Nitrite	0.8	" " ...	0.4	"	"
Nitrate	5.3	" " ...	25.7	"	"
Oxygen required.....	..	" " ...	6.0	"	"

"A study of the data tabulated above shows that greater nitrification is effected at Fort Myer than at Boston. The reduction in total organic nitrogen and free ammonia, and the increase in nitrate in the trickling filter effluent at Fort Myer are of greater magnitude than those obtained at Boston. The low free ammonia figure and the high nitrate figure in the Fort Myer trickling filter effluent are significant and indicate that a very satisfactory oxidation was taking place at the period of observation."

No extensive chemical or bacteriological work has been undertaken recently at any of the treatment plants. The monthly reports contain the following data: Total flow in million gallons per day; maximum and minimum rates of flow per 24 hours; amount of precipitation each day; temperature of sewage at inlet to plant; quantity of screenings and detritus removed; a daily statement as to the number of tanks in service; settleable solids (2-hour periods, cubic centimeters per liter) in raw sewage, tank effluent and final effluent; the quantity of sludge removed, in cubic yards; condition of filter beds; relative stability of the raw sewage and final effluent; and details relative to use of chlorine, etc. From these reports Table 1 has been compiled to show the results obtained in sedimentation and stability of filter effluents.

Table 2, from "Sewage Treatment", by Messrs. Kinnicutt, Winslow, and Pratt, page 164, is inserted here for the purpose of comparing the sedimentation capacities of the multiple-compartment sewage tanks and the Cameron septic tanks. Although the data given in Tables 1 and 2 are not directly comparable on account of the fact that the de-

TABLE 1.—Percentage of Removal of Settleable Solids by Tank Treatment and Relative Stability of Filter Effluents at Various United States Army Camps.

Data Compiled from Daily Records for 1919.

Camp.	January.		February.		March.		April.		May.		June.		July.		August.		September.		October.		November.		December.	
	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
Tank and Filter Treatment																								
Dix	98	†	97	†	97	†	96	55*	96	70*	95	95	96	98	95	95	97	95	97	97.7	99	98.4	98	93.3
Gordon	†	†	.....	....	91	89	95	94	94	100	96	99	97	100	.....	†	†	†	†	†	†	†	†	†
Pike	72	†	82	83	94	73	98	91	98	96	89	†	78	96	93	98	97	98	98	99	98	99	96	99
Fort Sheridan	.....	....	.....	....	.....	....	.....	....	†	†	†	†	96	62*	98	93	98	100	96	100	98	100	†	†
Tank Treatment Only																								
Lee	72	....	98	....	82	....	98	....	97	....	87	....	94	....	95	....	96	....	96	.....	95	.....	96	....
Meade	99	....	99	...	100	....	100	....	100	....	100	....	100	....	†	....	92	....	90	.....	98	.....	100	....
Merritt	94	....	96	....	94	....	93	....	96	....	97	....	95.5	....	95	...	97.2	....	97.2	.....	97	.....	†	....
Mills	†	....	†	....	†	....	†	....	98	....	98	....	97	....	95	....	96	....	†	.....	†	.....	†	....

A = Percentage of removal of settleable solids.
B = Percentage of relative stability.

* New filter.

† Data incomplete.

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terminations have not a common basis, nevertheless, it is quite evident from a study of these tables that there is a great advantage to be derived from sludge dams and baffles such as are used in the former tanks. Mr. Doten.

TABLE 2.—Removal of Suspended Solids in Ohio Septic Tanks.

Place.	Date.	Rate of sewage flow, in gallons, in 24 hours.	Hours. flow.	Suspended Matter.* Parts per million. Crude sewage.	Septic sewage.	Percentage of removal.
Ashland	July 11-12, 1906	150 000	6.3	70	50	29
Ashland	April 24-25, 1907	375 000	2.5	50	70	— 40
East Cleveland	June 26-27, 1906	365 000	11.2	210	400	— 91
East Cleveland	July 11-12, 1907	390 000	10.4	110	180	— 64
Geneva	June 21-22, 1906	181 000	5.2	60	65	— 8
Geneva	June 24-25, 1907	204 000	4.6	90	150	— 67
Kenton, N. District	October 5, 1906	18 000	24	220	180	18
Kenton, N. District	July 2-3, 1907	17 000	27	70	120	72
Lakewood	June 12-14, 1906	395 000	18	45	45	0
Lakewood	June 12-14, 1907	1 150 000	6.2	100	140	— 40
Mansfield	May 28-29, 1907	1 058 000	23	75	110	— 47
Marion	May 23-25, 1906	415 000	24	150	85	43
Marion	Nov. 8-9, 1906	370 000	27	90	140	— 55
Marion	April 9-11, 1907	578 000	17	40	45	— 12
Sandusky, Soldiers' and Sailors' Home	June 5-6, 1906	155 000	17	95	90	5
Sandusky, Soldiers' and Sailors' Home	May 8-9, 1907	174 000	15	95	90	5

* Suspended matter to nearest 5 parts below 100 and to nearest 10 parts above 100.

It is the writer's opinion that the importance of baffles in sewage tanks has not generally been fully appreciated by engineers. The results of tests covering long periods demonstrate conclusively that the baffled tanks provided with sludge removal pipe lines are much more efficient in sedimentation than the rectangular tanks, which are simply provided with one or two scum boards, and are not equipped with apparatus for the removal of sludge from the outlet end. In this connection the following quotation from Mr. Arthur J. Martin's recent inaugural address as President of the Association of Managers of Sewage Disposal Works in England, is especially interesting and significant:

"More pains might with advantage be taken to free a tank effluent from suspended matter before filtration. In a septic tank the partially digested sludge accumulates at the outlet end and as decomposition proceeds gives off a great deal of gas which breaks up the deposit and brings large quantities of it to the surface, where it is carried over with the effluent. This fouling of the effluent would not occur if outlets for sludge were provided at the outlet end of the tank and regularly used. By following precipitation tank practice one frequently finds the sludge valves placed at the inlet end."

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The data contained in Table 1 indicate that 5-ft. filter beds yield effluents of excellent quality. This would not be the result if the long period of dosing so commonly used in the design of municipal plants had been adopted. The humus from these filters is generally free from putrescible matter. In consequence, there is little gas disturbance in the secondary sedimentation tanks, with the result that the effluent is exceptionally free from suspended matter. The humus dries readily on drying beds and without odor.

Deep trickling filters, on the other hand, often produce humus which is decidedly putrescible and is objectionable on account of the bad odors which develop during the drying process. The odor factor is undoubtedly due to a large extent to the presence of decaying worms and other organisms which are deposited in the secondary sedimentation tanks. This condition is especially noticeable at the Fitchburg, Mass., plant, where the filters are 10 ft. in depth.

It is the writer's opinion that in future practice the local requirements as to degree of purification should be more carefully taken into consideration in designing the secondary treatment works. There are all gradations of such requirements, varying from the quality of a good tank effluent to that of the best possible effluent from a trickling filter or from the activated sludge process. It does not seem to be consistent, therefore, to install filters of the maximum capacity for oxidation and nitrification where a slight degree of purification would meet the requirements. As an illustration, a coarse rock filter, 3 ft. in depth, operating at a high rate per acre, would yield an effluent in some cases sufficiently stable to meet the local demands.

Where conditions do not admit of the construction of trickling filters for use in the secondary treatment of sewage it is believed that future practice will include designs of plants which incorporate the best features of both types of plants, sewage tanks with trickling filters and the activated sludge process. The cost of treating the effluent from tanks by the activated sludge process would be much less than to treat the entire sewage by that method. Such a combination plant would also have the advantage of making it possible to pre-treat the raw sewage before its entrance into the digestion tanks, so as to eliminate foul odors from stale sewage. The difficult problem of dealing with the large quantity of sludge from the activated sludge process would also be solved. This matter has been given considerable study by the writer, and plans are now under preparation for developing a plant of this type, in order to deal with problems where such conditions as limited area, proximity to habitations, and high degree of purification are controlling factors. However, the writer wishes it to be distinctly understood that he considers the tank and trickling filter treatment of sewage the most practical and efficient method for general use at the present time. It is only by the adoption of the best elements

Mr. Doten.

and the elimination of the impracticable features that practice in this field of engineering may be brought to a high degree of perfection and standardization. Cost of construction would then be lower and operation more uniform and efficient.

The problems in sanitation dealt with during the war by the writer have been very interesting. In this work he has been ably assisted by engineers of broad experience and by those having special qualifications for the service. The writer refers particularly to the following engineers whose services have been of exceptional value: J. F. Brown, E. B. Besselievre, and A. L. Sherman, Associate Members, Am. Soc. C. E., and Messrs. H. R. Abbott, R. P. Hough, R. M. Ingham, Edgar B. Kay, and R. M. Renwick.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

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Paper No. 1433

ADDRESS AT THE ANNUAL MEETING JANUARY 15TH, 1919.

By Arthur N. Talbot, President, Am. Soc. C. E.

If the President's address had been given at an Annual Convention, as the Constitution prescribes, a theme of greater entertaining value might have been chosen, but at the Annual Meeting it seems fitting to review some of the happenings of the year and to discuss the outlook for the Society.

War Service.

The year has seen the nation engrossed in the great World War. As engineering and the engineer have played a vital part in war activities, it was to be expected that the membership of the Society would take an active and responsible part. The summary of the Society Membership serving in the armies and navies of the United States and its allies, as reported in the *Proceedings* of the Society, is indeed a glorious record. A grand total of 1 640, ranging from major-generals and rear-admirals down through to the ranks, with a very high percentage in the positions of great responsibility, tells a story of active patriotism. To this should be added the large number of members who, at a sacrifice of financial and home interests, have given up their time to governmental and war activities in a civilian capacity, and others who have taken responsibilities in war service in various ways. The response made by the engineer and the service he has given prove that he has attempted to do his part in full measure.

The achievements of the engineer in war activities here at home are greatly to his credit. The incomplete accounts of the work of the American engineer on the soil of France at points of debarkation, on lines of communication, and at the front present a wonderful story; and the award of the Distinguished Service Cross and the French Croix de Guerre to many of our members tells of their great valor on the field of battle. Nor can we forget the many who sacrificed their lives in this great struggle—their's will be bright stars in the flag of humanity. We rejoice that right has prevailed, even at a great cost of effort and of lives. We trust that the conditions of peace settlement will be a guaranty of the future rule of justice and democracy.

Membership.

Notwithstanding the distractions occasioned by the war and the call of engineers to army and government service at home and abroad, the growth of the Society has been about equal to the average of recent years. The total membership on December 31st, 1918, was 8 933. As has been the case each year for several years, the number of Juniors has decreased during the year, the transfers to higher grades and the losses by death, resignation, and dropping being greater than the accessions.

Although the growth of the Society has continued regularly for many years, it may be noted that the total membership is exceeded both by the American Institute of Electrical Engineers, with a membership of 9 668, and the American Society of Mechanical Engineers, with a membership of nearly 10 000. Rapid gains have been made by these two societies in recent years. It may be noted that some societies, including many local societies, have secured large increases through membership drives, while the policy of our Society has been, not to encourage membership campaigns, but, instead, to accept the smaller but steady and substantial growth which comes without solicitation, in the belief that this method gives character to the Society and is productive of its best development.

Financial Matters.

Although the expenditures properly chargeable to the work of 1918 are considerably less than the Society's income for the year, the treasury is low. There are a number of reasons for this. The cost of the alteration of the Fifty-seventh Street property has taken about

$18 000 in addition to money secured on the mortgage of the property. As there was no income from the property until late in the year, the interest on the mortgage was an added expense. The annual charge to the Society for space in the Engineering Societies Building, together with the Library assessment, amounts to about $10 000 more than corresponding expenses in the old location. Rising prices have also affected expenses. The remission of the annual dues of many members in war service reduced the income for the year. It will be recalled that although a balance was reported a year ago, the printing and binding of the *Transactions* for 1917 had not been paid for. Available funds had been used in completing the payment of the Society's share in the extension of the Engineering Societies Building, and this is the real cause of the deficit. Efforts have been made during the year to keep down expenditures and to restrain activities. In a general way it may be said that, leaving out of consideration the alteration of the old Society House, the income for the year has been about $14 000 more than the expenses, and that, if the bill for printing and binding the 1918 volume of *Transactions* is considered and the special funds taken into account, the deficit on January 1st may be called $20 000, counting the advance dues collected as income for 1918.

The Fifty-seventh Street Property.

It will be recalled that it was announced a year ago that the property of the Society at 220 West Fifty-seventh Street, which had been the home of the Society for twenty years, had been given over temporarily to the use of the New York Federal Food Board, without charge to the Food Administration, except for the cost of maintenance of the building. It seemed desirable to make the property a source of revenue, and this involved changing the building from a society house to a business building. The remodeling has gone on since August, a lease for a term of ten years has been executed, and the building is now occupied by a responsible business firm. It is expected that the net annual returns from this property will be in the neighborhood of $10 000, not counting renewals and depreciation. The property itself is estimated as worth upwards of $400 000, and there is an encumbrance of $200 000. The value of land in that part of New York City is said to be rising rapidly.

The Engineering Societies Building.

The Society has now occupied its quarters in the Engineering Societies Building for more than a year. The work rooms of the office force have been found convenient and satisfactory. The reading room has been maintained and made attractive for members. The Society Library was merged with the Engineering Societies Library, and the library facilities of members are in reality greatly enhanced. The Society contributed as its share of the cost of the enlargement of the Engineering Societies Building the sum of $262 500, the whole property now being estimated as having a value of $2 500 000. Removal to the Engineering Societies Building has brought new associations, and particularly a closer association with the other Founder Societies. From every side come expressions of approval and satisfaction. In joining quarters with the other societies it is felt that a forward step was taken in extending the influence of the Society and the profession of engineering. It is not too much to hope that the coming together in the one building will prove to be a forerunner of many mutual undertakings and of much co-ordinated and co-operative work, all helping in extending the influence and promoting the solidarity of the profession.

Meetings.

An effort has been made during the year to get away from the usual programme of engineering papers, and, as far as possible, to give at the semi-monthly meetings matters relating to war activities. A number of interesting talks and addresses were given by American engineers and army officers, as well as by British and French officers. Naturally, the proper conduct of a war imposes restrictions regarding giving publicity to war activities, and on this account important war developments of engineering interest were not available for the use of the Society. An important development in the conduct of the meetings of the Society was the creation of a New York Meetings Committee charged with the conduct of the second meeting of each month. This act gives resident members a responsibility and an opportunity to make this particular meeting of special interest to local members. It is hoped that the movement will prove to be of great significance to the Society. Because of the war the Annual Convention was again abandoned.

Publications.

With the progress of time the war has absorbed more and more of the energy of the country; this condition has been reflected in the Society's *Proceedings*. As a measure of economy, also, the publication of papers has been curtailed. Six papers only were printed in the first half of the year; only discussions of papers have been given in the last half. On the other hand, the space devoted to announcements, items, and reports has been increased, and an attempt has been made to include more and more items of interest to members. The *Transactions* for 1918 are in press and will be issued soon.

Special Committees.

War conditions are not favorable for technical committee work. Committee members have been engaged in a variety of war activities, and, with all the distractions of the year and the scarcity of assistants, it would not be strange if the reports indicate less than normal activity. With the return of peace conditions it may confidently be expected that the Society's activities through its special committees will assume proportions in keeping with the opportunities which this agency gives for advancing professional knowledge and improving the standing of the profession, which should mean an expansion of committee work.

American Engineering Standards Committee.

A matter of great possible significance is the formation of a permanent organization, known as the American Engineering Standards Committee, to carry on the important work of co-operating in engineering standardization throughout the entire field of engineering standardization. The organization has been formed by the following societies: The American Society of Civil Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and the American Society for Testing Materials, with three representatives from each of these societies and from each of the following United States Departments: Commerce, War, and Navy. Other societies and bodies may be allowed to co-operate in the work, by becoming "co-operating societies", but are not to have representatives on the Standards Committee. The procedure for formulating standards is somewhat com-

plicated. Any interested organization may propose the development or revision of engineering standards in any engineering field, and, if the Standards Committee approves, it will designate one or more of the co-operating societies as sponsor or joint sponsors for the standard under consideration, and give the sponsor authority to organize a special committee, to be known as a sectional committee, to carry on the work. Sectional committees dealing with standards of a commercial character (specifications, shop practices, etc.) shall be made up of representatives of producers, consumers, and general interests. Special committees dealing with standards of a scientific or non-commercial character shall consist of persons specially qualified, without regard to affiliations. The special committees will report their recommendations to the sponsor, and the sponsor, on approval, to the main Standards Committee. The Standards Committee may then approve the proposed standard as "Recommended Practice" or as "Tentative Standard", and when, in the opinion of the Standards Committee, after approval by the sponsor, it has proved its suitability, the Standards Committee may constitute it an "American Standard."

It will be seen that the organization may deal with standards and specifications in the whole range of engineering: materials such as steel, timber, cement; articles such as beams and structural parts, rails and ties, water pipe, machine parts; methods of work such as road building, concrete making; governmental requirements such as building regulations, requirements for working stresses, regulations for fire-proofing; scientific standards, such as methods of testing, standard technology, requirements for water supply; and specifications and standards of hundreds of kinds. It thus offers great possibilities for usefulness, and there are sources of danger. It is also to be seen that the formation of this organization may take from the Society opportunities for independent action on some matters. If the American Society of Civil Engineers handles its part in this organization vigorously and wisely, it will prove an opportunity for service to the profession and the public. If it does not care to be active in the matter, the field will be taken by others.

Engineering Societies Employment Bureau.

On the initiative of the Board of Direction of the Society, an Engineering Societies Employment Bureau has been established by

United Engineering Society, with the Secretaries of the four Founder Societies as the Board of Managers. It is believed that this plan will prove helpful to members and to others.

United Engineering Society and Related Organizations.

The past year is the first full year in which the Society has participated in the co-operative activities of the Founder Societies, and it may be well, therefore, even if a repetition is made of information published elsewhere, to review briefly the organization and functions of United Engineering Society, Engineering Council, Engineering Foundation, and other related organizations of the Founder Societies. Since the Society is participating in the work of these organizations and has given over to them some functions formerly carried on within the Society, the activities of these organizations should be considered as part of the work of the Society. United Engineering Society is the central organization. The related organizations, such as Engineering Foundation, are established and maintained through or by the United Engineering Society. United Engineering Society, is the holding corporation for the real property; it holds the title to the Engineering Societies Building. United Engineering Society also holds the title to such trust funds as may be contributed for the special purposes of the related organizations. United Engineering Society maintains the building and determines the apportionment of the budget among the several Founder Societies and the associate societies which occupy space in the building. It may establish departments or related organizations. United Engineering Society is thus the business and co-operating corporation which acts for the four Founder Societies. The governing board is the Board of Trustees, which consists of twelve members, three being selected by each of the Founder Societies.

United Engineering Society maintains and conducts the joint engineering library through the Library Board of twenty-one members, composed of the Director of the Library, the Secretaries of the Founder Societies, and sixteen other members, four being designated by each Founder Society.

The department known as the Engineering Foundation was established for the furtherance of research in science and engineering and for the advancement in any other manner of the profession of engineering and the good of mankind. The control and administration are placed in

the Engineering Foundation Board which is composed of the President of United Engineering Society, *ex officio,* and the following members elected by United Engineering Society: one trustee of the United Engineering Society from each Founder Society, two members from each Founder Society, nominated by its governing board, and three members at large.

Engineering Council was established "* * * in order to provide for convenient co-operation between the four Founder Societies, for the proper consideration of questions of general interest to engineers and to the public and to provide the means for united activities of common concern to engineers. * * * The Council may speak authoritatively for all member societies on all public questions of a common interest or concern to engineers, unless objection be made by a majority of the representatives present of one of the Founder Societies or by one-fourth of the representatives present and voting," which is the phraseology of the By-Laws of United Engineering Society. It has been the general understanding that Engineering Council was formed to undertake, not the technical activities of professional societies, but rather the non-technical, semi-professional, and general work, and especially that which bears upon public interests and public affairs. At a recent meeting, Engineering Council, in an effort to define its field, declared by resolution as follows:

"That Engineering Council understands its fields of activity to be approximately as follows:

"(1) Council may deal with any matter of general interest for which joint action of two or more of its member societies would have been appropriate, if Council had not been established.

"(2) Council may initiate and carry through projects of the general character defined in the by-laws, for which the necessary financial provision has been made; but Council shall not undertake expenditures in excess of appropriations for its uses made by United Engineering Society on behalf of the Founder Societies and the contributions from other member societies, unless specific provision shall have been made therefor by subscription, donation or otherwise; moneys received by Engineering Council shall be turned into the treasury of United Engineering Society and disbursed by it, for the purposes designated.

"(3) Council may take up, and in its discretion act upon, any matter of general interest referred to it by any member society or by any other society—national, state or local—or by any branch of Government, or by an individual or group of individuals."

It further declared:

"Engineering Council will, as a rule, avoid considering any matter which is specifically within the province of only one member society and not of others."

It is evident that Engineering Council has broad powers and responsibilities in the co-operative field.

It may also be proper to make note of some of the activities and accomplishments of these organizations. As the addition to the Engineering Societies Building was completed in 1917, the work of United Engineering Society has been the maintenance of the Engineering Society Building and the carrying on of relations with the Founder Societies and the related organizations. At the beginning of 1918 an important change in organization was made by the engagement of a Permanent Secretary who also acts as Secretary of Engineering Foundation and Engineering Council. Alfred D. Flinn, M. Am. Soc. C. E., the Permanent Secretary, entered upon his duties on January 1st, 1918.

The Engineering Foundation was organized in 1916 when Mr. Ambrose Swasey, of Cleveland, Ohio, Past-President of the American Society of Mechanical Engineers, offered the United Engineering Society $200 000 as the nucleus of an endowment "for the furtherance of engineering, for the advancement of the profession and the good of mankind". This was a magnificent gift for a great cause—the advancement of engineering and the good of mankind. This eminent engineer, however, did not rest content with one contribution, but in September, 1918, he added $100 000 to the endowment, making the total $300 000. It is confidently expected that the good example set by Mr. Swasey will be followed by contributions from many other persons—engineers and others—including the smaller contributions from those who cannot give large sums, as well as princely gifts from those of large means.

The income from the present endowment, of course, would not permit the Engineering Foundation to undertake engineering research itself on any considerable scale, for the carrying out of the larger engineering research activities requires large sums for successful accomplishment, and the engineering field is very large and the problems are very numerous. The Engineering Foundation wisely decided to devote its main effort for the present to activities which would serve to

encourage, direct, and develop research agencies and activities, at the same time giving such aid to research work as is found practicable.

The possibility of the United States being drawn into the World War brought about, in the summer of 1916, the organization of the National Research Council, an organization of scientists, engineers, and educators. The Engineering Foundation saw the needs of the organization for funds, and contributed facilities and money until Government support became available for the work. Thus started, the organization became extremely helpful to the Government in the prosecution of the war. Finally, it became the Department of Science and Research of the Council of National Defense, and, by executive order of May 11th, 1918, it was permanently established in its useful work. The Engineering Foundation was very helpful at a critical time in thus assisting in an important movement for stimulating research, and thereby rendered the greatest possible service to the country at the time. With the co-operation of Engineering Foundation and the National Engineering Societies, the National Research Council early formed an engineering committee, which later became the Engineering Division and has the following aims:

"To carry out the general purpose of co-ordinating the scientific resources of the entire country as regards engineering and to secure the co-operation of all engineering agencies in which research facilities are available."

The Engineering Foundation and the American Society of Civil Engineers have representatives on the Engineering Division. The Engineering Foundation has continued to affiliate with the National Research Council in various ways, as, for example, in collecting information about the several hundred industrial research laboratories existing in this country and in helping to stimulate industrial research.

It would give a wrong impression if reference were not made to a number of instances of stimulation of research on special problems by small grants of money, such as for the investigation of the secret selective control of wireless communications, the wear of gears, weir measurements of water, and a proposed method of camouflaging vessels. Generally, however, Engineering Foundation is proceeding on the policy that its resources should be devoted to large enterprises and the solution of problems of broad application, rather than to detailed investigation of limited problems. The opportunities of Engineering

Foundation in helping in the advancement of engineering are abundant, for the advancement of the art of engineering is dependent on scientific research, and the standing of the engineering profession as a profession will be markedly affected by its attitude toward research.

Engineering Council has started out on important work. Considerable attention has been given, since its organization meeting on June 27th, 1917, in exploring its proper field of activity, in deliberating on what it may and may not undertake, and what it should and should not undertake—important questions for a body which is starting on undeveloped territory. At the same time, it has gone ahead with activities. Through the American Engineering Service, which it organized for war-time service, it rendered aid to the Government, resulting in procuring thousands of engineers for the war, navy, and other departments. Through the War Committee of Technical Societies, assistance was given the Naval Consulting Board and the Army General Staff in examining thousands of suggestions for naval and military devices. The names of the committees of Engineering Council at work indicate in some measure present or intended activities: Public Affairs, Fuel Conservation, Patents, Industrial Affairs, Water Conservation, for all of which progress is reported. At different times Engineering Council has been represented at Washington in respect to matters of interest to engineers, and its activities there have borne fruit. It is organizing a National Service Committee to give instant, active, and constant representation of engineers at Washington, especially on matters relating to reconstruction affairs. Engineering Council is endeavoring to keep in communication with communities of engineers all over the country. The preliminary work of organizing its activities has made considerable progress during the year. Engineering Council has an important task in attempting to provide for consideration of matters of common concern to the profession.

It will be agreed that the aims and functions of United Engineering Society and of these co-operative agencies are of far-reaching importance to the American Society of Civil Engineers and its membership. Their opportunities and responsibilities are indeed large.

The Award of the John Fritz Medal.

The profession of civil engineering has been recognized and honored through the awarding of the John Fritz Medal to a prominent and

distinguished member of the Society, J. Waldo Smith, for his most distinguished service in engineering and great achievement as engineer in providing the City of New York with a supply of water. The public presentation was made at a meeting which took the place of a regular meeting of the Society. It is worthy of note that, in the fourteen times the medal has been awarded, this is the second award for what in the narrower sense may be called strictly civil engineering achievement, the first recipient being the late Alfred Noble, Past-President, Am. Soc. C. E.

A Mission to France.

Responding to an invitation extended to the Society by the French Society of Engineers, with the approval of the French Ministries of Public Works, of Commerce, and of Armament, to send a small delegation of engineers to meet in joint engineering congress in Paris for the purpose of discussing technical matters relating to reconstruction work and the utilization of commercial ports, the development of navigable waterways, the development of water power, the improvement of road systems, and of other engineering and industrial interests of France, six representatives of the Society were chosen, who, with three representatives appointed by the other Founder Societies, sailed for France early in December, and are now engaged in sympathetic assistance in the consideration of important problems related to the future development of the resources and opportunities of our sister republic. The State Department sanctioned the going of the mission, in the belief that its presence in France would be taken as evidence of America's desire to make real the ideals that have triumphed in the war. The matter is significant, not only because it gives American engineers an opportunity to be of service to the engineers and people of France, but because of the opportunity it gave the Society to extend to the other National Societies an invitation to participate in a matter of general interest, an action which has been accepted by the other societies as an evidence of our desire to co-operate.

The Committee on Development.

At its June meeting, the Board of Direction took a step which it is hoped will have significant and far-reaching results in the development of the Society and in increasing the usefulness of the engineering profession—the creation of a committee to report on the purposes, field

of work, scope of activity and usefulness, organization, and methods of work of the American Society of Civil Engineers—the purport of the resolutions being amplified by the preamble. The method adopted for selecting the committee gives a wide geographical distribution and an opportunity for the representation of local views. The Committee on Development held its first meeting in Chicago in November, spending three days in the discussion of its work. The preliminary report of the committee has been sent out to the membership. Sub-committees are at work on the development of the subject, and doubtless they will find a large task ahead of them. To prepare a final report adequate to the problems set before the committee and worthy of so great a cause will take long hours of work, much thoughtful deliberation, and the highest constructive powers. Let us give our full support to the Committee on Development in its labors.

It is worthy of note that the three other Founder Societies have appointed committees to undertake the same kind of an inquiry.

The Work of an Engineering Society.

It would be presumptuous to attempt to forecast the outcome of the work of the Committee on Development. It seems proper, however, to try to bring to your attention this morning, in so far as the time will permit, some of the matters which appeal to me in studying what should be undertaken by the American Society of Civil Engineers.

What are the purposes and functions of an engineering society such as the American Society of Civil Engineers? In discussing this question, it is necessary not to restrict ourselves to the purposes given in the Constitution of the Society, which may merely follow precedent or may be worded to conform to a State statute governing the charter of the Society. Whatever may be our views of details, may it not be agreed that at least the following should be involved in a national engineering society:

A learned society; a collector and disseminator of engineering knowledge; a forum for the discussion of engineering questions, simple and complex, small and large; a stimulator of research and of the progress of engineering science; a developer of engineering methods and practice; an active force within and without professional circles; a creator of engineering policies and an originator and patron of constructive and progressive movements in engineering; an opportunity

for meeting and knowing the engineers of the country; a means of mutual helpfulness and co-operation for the profession and its clientele; an instrument for advancing both professional interest and public welfare; an agency for discussing the various phases of public problems and for participating in the service which all thinking organizations should give to society.

And the engineering society has justified its existence. It has helped to create the engineering profession. Its accomplishments, technical and extra-technical, all redound to its credit and to the credit of its membership. To recount the activities and achievements of the engineering society, within its walls and in service to the public and to society, in matters that are known to all, and in ways that are not usually recognized, would consume much more time than is available here. And the engineering society has given *esprit de corps* to the profession. It has made its members feel self-respecting. It has called to the attention of many the high standards held by the profession and the responsibility attached to membership in it. It has continuously raised the standards and the standing of the engineering profession, and has assisted in establishing this standing in the eyes of the public. The members who put the most into their membership relations get the greatest benefit, but the inactive member, perhaps unconsciously, profits by its beneficent influence, for there are intangible benefits and advantages which come from the presence and activity of the engineering society.

All this is accepted without argument in an audience such as this. The question up for consideration is what should be done to extend the usefulness of the engineering society and to make it a greater force for its membership, the profession, and the public.

Technical Activities.

The technical side of the activities of an engineering society is of prime importance. Engineering is and must continue to be a technical art, a technical business. Its body of technical matters is far larger and more important than existed a generation ago, and the requirements for technical knowledge in engineering work are increasingly important. By technical matters are meant more than mere mathematical methods and the routine work of engineering; all the relations to the science and art of engineering in and out of professional circles are included, and

also the technical features of the business side and the technical relations with the public. Important as is the administrative and executive side, the characteristic and determining feature of engineering is the technical side. The time is past when ingenuity, rule-of-thumb methods, and workmanlike skill alone will make an engineer. If engineering is to be only a combination of business and routine construction, it becomes a craft, a mere line of business. The engineering society should be a principal factor in the development of the science and art of engineering, in developing and recording the technical side of engineering, the scientific substructure, the overlying elements of practice, the variform connections with business life. By the technical side is here meant, not alone a knowledge of engineering principles and facts; it involves creating engineering policies, developing engineering practice, improving and controlling engineering connections with the outside business world. To carry out its technical functions properly is a high aim for an engineering society.

One of the methods of carrying on this technical activity has been through the presentation and discussion of papers and publications on engineering topics. The contributions of the Society in this direction have been of great importance. It would seem worth while, however, to raise the question whether the Society is doing all it should do in this direction. Is there not need for papers in fields not often taken up in the Society *Transactions?* Should there not be more short, terse papers, devoid of details, which should stimulate and set engineers to thinking? Would it not be possible and desirable to have more papers dealing with principles, with the advances of engineering, with new methods and requirements? May it not be worth considering whether we should hold to the long-used policy of limiting ourselves only to those papers which are offered to the Society from time to time, regardless of whether this policy leads to ignoring large fields and results in transferring to other societies and agencies activities in these lines; and, aside from papers, are there not opportunities for dissemination of knowledge which may properly be undertaken—like reviews of engineering construction and methods, digests of important engineering literature, which seem now not to have adequate expression through other agencies?

Work of Technical Committees.

Technical committees form one of the most effective means of carrying on the technical activities of an engineering society. Such

committees are able to utilize the experience and ability of many members and to take advantage of facilities and opportunities afforded by outside organizations. They can co-ordinate and unify practice, develop methods and recommend acceptable practice, acquire technical knowledge, formulate principles and standards, and recommend improvements and advances in the art. Their influence can reach out far beyond the bounds of the Society. Experience shows that committee work may be a power for good. Two of the specialized engineering societies of national character carry on the larger part of their work through committees. The American Railway Engineering Association has 23 technical committees, with a combined membership of 480 out of a total membership of 1 300 in the Association. The subjects for these committees range from ballast and rail to economics of railway operation and economics of railway labor. The American Society for Testing Materials has 38 technical committees and 144 sub-committees, with a combined membership of 1 094 out of a total membership of 2 300. Committee A-1 on Steel, which has had great responsibilities in the past few years, consists of 121 members. The membership of both of these societies gives a great amount of time and attention to committee work. The influence of the work of these two societies has gone out far beyond their membership, far beyond the limits which the names of the subjects would seem to make possible. At the risk of seeming disloyal, the opinion may be advanced that in influence on engineering practice, in effect in the industries, in influence on relations of engineering business, in advancement of engineering knowledge and the methods of manufacture and construction, and in influence in the outside business world, the achievements of these two specialized societies in the past few years have been such as to make some of the National Engineering Societies look to their laurels for leadership. It may not be too much to express the belief that the Committee on Development will find the need and opportunity for technical committee work an important subject for study.

Specialized Divisions or Sections of the Society.

What seems to be an opportunity for energized activities lies in the formation of specialized divisions or sections of the Society to give attention to special fields of work, such as railway engineering, structural engineering, and hydraulic engineering. Members would enroll in sections in which they are interested. Sessions and conventions

would be held to discuss matters of common interest. The railway engineering division might hold membership in and co-operate with the American Railway Engineering Association, giving attention to certain features of the convention and committee work of that society. Another division might affiliate in some way in the work of the American Society for Testing Materials—in fact there is an unoccupied field closely allied to the work of that society. A division interested in sanitary engineering might find a welcome with the American Public Health Association. The activities of such specialized divisions would reach outside the membership and give new points of contact and influence over the outside world. New forces would be set up among the membership, and new opportunities for participation given.

Formulation of Engineering Policies.

Both the committee work and the specialized divisions constitute effective agencies for creating engineering policies and originating constructive and progressive advances in engineering and related matters—important functions for an engineering society.

Local and Regional Organizations.

In any discussion of National Engineering Societies it must be borne in mind that the membership is scattered over the country; that there are numerous centers of engineering, and that these are yearly becoming more important; that local engineering societies in these centers are essential elements in the life of the engineering profession; that these local societies furnish a means by which the young and growing man meets and knows other engineers and gains opportunity for development; that the National Societies may be greatly strengthened and their influence extended by closer affiliation with these local societies, and that any thorough consideration of the subject of aims and organization should endeavor to find ways and means by which the National Society may best effect closer co-operation with the local engineering society. It is also true that local conditions vary greatly, from place to place, over the country, and that the problems and the methods of contact with the public are far from uniform.

In the last few years there have been formed twenty-two local associations of members of the American Society of Civil Engineers (a term which I hope will be replaced by a less clumsy one such as

Chapter) at centers of engineering over the country. These local associations, if cherished a little, may be expected to be very active elements in the life of the Society. They should also become important elements in the life of the communities in which they are.

In many of the centers of population the local engineering societies are well established; in others they will become so. It would seem wise, in any attempt at organization of the profession throughout the country, for all the National Societies to join forces with these local societies in every possible way. In every center of engineering the local engineering society should be the basis of organization. Some plan should be devised, perhaps by increase of dues, to contribute revenue from the treasury of the National Societies to the Chapters for use in this union work with the local societies. It is evident that closer connection with the local engineering societies will promote loyalty in the National Societies. I believe some form of organization may be worked out which will win the support of the tributary membership and greatly strengthen both National and Local Society.

For certain purposes it may be found desirable, also, to have a regional organization, covering territory corresponding to districts, like those of the geographical districts used in the selection of the Nominating Committee, for handling matters of common interest over a region, a matter which could be worked out in connection with the management of the local associations.

In dealing with our distributed membership, the situation in New York should not be overlooked. The provision for a New York Meetings Committee to take charge of the second meeting of each month, made by the Board of Direction on the recommendation of the Secretary, is a step in advance. It would appear to me that, in any new plan for local associations or regional organizations, the resident membership (those within fifty miles of the New York City Post Office) should be granted some degree of autonomy, by the formation of a local association or chapter or otherwise, and that the New York membership should be set at work and stirred up to realize and utilize the opportunities and facilities at their door.

Relations with Other Societies.

Intimately connected with the question of local associations and local societies is the question of our relation with other National

Societies. These relations are yearly becoming of greater import. The four Founder Societies have many activities in common. Engineering Council may soon have its membership increased by three more societies. The Engineering Standards Committee will have the co-operation of a number of societies, besides the four Founder Societies. The opportunities for co-operation are increasing. It is increasingly apparent that engineering is a single profession; the various branches ramify and merge into each other in numerous ways. Many engineers think there should be one large inclusive engineering society to take the place of the several National Societies—to be made up of the 35 000 engineers in the four Founder Societies or the 100 000 or 200 000 who may claim the right to be classed as engineers. It does not seem probable now that the immediate future will bring about such a single organization. In default of this desideratum, the best use possible must be made of existing agencies. Organization should be effected which will bring united effort along many lines. The further union of National Societies and their co-operation with the local societies of the engineering centers in developing engineering society activity are potential factors for the future. This, too, is a fruitful field for the Committee on Development.

Relation to Public Affairs.

It has frequently been remarked that the engineer is likely to be confined so closely to his own work that his influence does not extend out as far as it should, and particularly that he has not taken part in public affairs, in governmental and economic activities, to an extent warranted by his training and business standing; and, further, that the engineering society has not accepted in full measure its opportunity and its responsibility in public and civic matters. To recount the accomplishments of the engineer in civic affairs, in governmental activities, in influence on public policy, in the solution of industrial problems, would show a creditable record; and the engineering society likewise has achievements greatly to its credit; but, notwithstanding all these accomplishments, it must be apparent to all that the engineer and the engineering society now have opportunities and responsibilities in the direction of public duty far beyond what they have hitherto considered as their share. This is especially true at the present time, when the successful solution of so many public problems lies within the province of the engineer.

The consideration of quasi public matters has not always been accepted as a proper field for Society activity. As an illustration, the opposition to the creation of a Committee on Valuation of Public Utilities may be recalled. When the Board of Direction was discussing the measure, a member opposed it because he felt strongly that the engineer should confine himself to the design and construction of engineering works, and another because valuation was not an engineering question and should be left entirely to the Courts, overlooking the service which only the engineer could give in correlating the engineering data and establishing principles to be considered in making decisions. Fortunately, the advocates of the appointment of the committee prevailed, in the end, and a magnificent report and discussion was given to the public, a report and discussion which doubtless will continue to be used and quoted whenever and wherever the valuation of public utilities is discussed by layman or Court. Besides, the activities of the committee stimulated thought and provoked discussion throughout our membership as few subjects have done.

How the engineer and the engineering society may best deal with public matters is a complicated question. The engagements of most engineers are not like those of the lawyer. There is a great diversity of condition of service and environment, even in the same lines of engineering, and the interests are very diverse. Evidently, a satisfactory plan may be evolved only after much study, and only a few of the elements of the situation may be touched upon here.

Part of the public work of an engineering society lies in bringing to the attention of public bodies technical and other matters already available through its committees and its publications, and in giving co-operation along lines where general technical information and direction will be helpful. Civic improvement and city planning, health and sanitary conditions of cities and communities, industrial conditions and their betterment, the need of high-grade engineering supervision of public work, are examples in one direction only. The public should be taught that there is a great body of knowledge, experience, and expert judgment the utilization of which greatly benefits mankind.

Certain matters of public interest are frequently considered through independent or auxiliary organizations, more or less temporary in character. The agricultural interests use such agencies rather freely. The

Reconstruction Congress of War Service Committees is a recent activity, held under the auspices of the United States Chamber of Commerce, to recommend what American business ought to do for the Government and what the Government ought to do for American business. Waterway conventions and drainage congresses are also examples of this method of work. Unless savoring too much of propaganda and special interests, the method of disseminating information, stimulating discussion, and influencing public opinion through special organizations, is effective and legitimate.

Direct connection with Governmental agencies may be difficult. Special instruments may need to be created. Great credit should be given to Engineering Council for the start it has made in handling public matters in this first year of its organized activity. Its recent creation of a National Service Committee, with an appropriation for expenses, for active and constant representation of engineers at the seat of the National Government, in these times when reconstruction problems involving engineering are at the front, is a very promising movement. It illustrates the type of work which the engineering society can undertake. A variety of methods will have to be used. Those public problems which are local, or are confined to States or limited territory, may be handled most satisfactorily through local or regional organizations.

It would be premature to try to outline here methods for accomplishing the great work in public matters which should be undertaken, or the form of organization which is needed. The formation of a large and representative public affairs committee has been suggested, and also a delegate convention organized to discuss matters of public interest and intended as a forum for voicing the sentiments of the organization. More than these are needed. So much time has already been taken in this address that I will not attempt to outline my views. Whatever is done must give opportunity for exercising initiative and providing powers for action. Leaders must then energize the machinery provided.

Financial Considerations.

The activities here referred to will require adequate funds; it takes money to do things in this world. Our equity in the Engineering Societies Building, which seems a princely one, can never be a source of revenue, but must continue to be a large item of expense, however

much it adds to the standing and respectability of the Society. The present stringency in our treasury is believed to be temporary, and January, 1921, should begin with current expenses all paid and a balance in the treasury at least equal to the advance dues collected, together with such special funds as may have been contributed from outside sources.

It must be understood, however, that if the activities of the Society are to be extended, funds must be provided to pay the accompanying cost.

Part of this may well be taken from the annual balance of $20 000 or $30 000 which in normal years may be left after paying the expenses of present activities. It seems reasonable that a society which has become established financially should not continue to put aside from annual receipts very large sums as endowment; the increase in endowment may be expected to come from other sources.

Increase in membership will bring increased revenue beyond the accompanying expense. Many believe that there should be a considerable increase of membership.

With increased activities and higher costs for everything, it is possible that an increase in annual dues may be worthy of consideration.

The expense of some activities may be borne from contributions from various sources, probably a fruitful source of revenue for several lines of activity.

Much of the work which may be undertaken will be carried on by individuals and organizations—the Society's part will be to organize, stimulate, and direct.

If it is agreed that there are activities which the Society should undertake for the benefit of the profession and society, activities that will bring results and that will help in making the Society a force of greater and greater influence, who will doubt that resources will be found to finance the work?

Conclusion.

Altogether, it is a great problem—one worthy of the best thought of the Committee on Development and the Society. To shun the visionary, impracticable schemes, and yet be not too complacent about past accomplishment; to seek to be national and not provincial; to try to be both democratic and efficient; to lead in the progress of the times and

yet to hold to the old and proven good; to work to be a powerful engine in technical affairs and a vital force in the life of the profession; to learn how best to give service to the community and the nation; to provide methods by which the profession may lead in progressive movements—these are ideals worth striving for in making a plan. It should be borne in mind, too, that elaborate or complicated organization is likely to be ineffective; the machinery is not self-propelling. Mere paper proposals will not suffice. Whatever is done, we must keep down on earth, accept only workable plans, but still not be content with stagnation or mediocrity. It is my belief that a plan may be worked out befitting the needs of a great profession, and that, with such a great opportunity before it, the Committee on Development will evolve a plan satisfactory to the Society. The discussion of the whole question will serve to stimulate interest, improve spirit, and arouse latent vitality. May we not expect that the result will be a stronger, more useful, more virile Society? As members we confidently look forward to a great future for the American Society of Civil Engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1434

THE PROPOSED NEW YORK AND NEW JERSEY VEHICULAR TUNNEL

AN INFORMAL DISCUSSION AT A MEETING OF THE SOCIETY, MARCH 19TH, 1919.

BY MESSRS. EDWARD A. BYRNE, JOHN F. O'ROURKE, PAUL G. BROWN, J. V. DAVIES, JAMES FORGIE, W. C. PARMLEY, T. KENNARD THOMSON, F. LAVIS, CALVIN TOMKINS, AMOS SCHAEFFER, AND E. E. R. TRATMAN.

Mr. Byrne.

EDWARD A. BYRNE,* M. AM. SOC. C. E.—The purpose of this discussion is to call attention to some of the reasons for constructing a vehicular tunnel to connect New York and New Jersey, and to discuss the merits of the various types of tunnels proposed.

Before proceeding, however, it might be well to state that, although the construction of a bridge across the Hudson River had been agitated for many years, it was not until 1906 that any official action was taken by the States of New York and New Jersey. In that year both States appointed Bridge Commissions to report on the advisability of building a bridge across this river. In 1913 additional powers were granted to the New York Commission by the Legislature, which enabled it to direct its attention to tunnel construction, the New Jersey Commission having received like powers from the Legislature of that State.

In February, 1913, Messrs. Jacobs and Davies reported to the Commissions that a tunnel was entirely feasible, and recommended two tubes, to be driven by the shield method—one for east-bound traffic and one for west-bound—each with a roadway to have a width of 17 ft. and head-room of about 12 ft. 6 in., the inner diameter of the tunnels

* New York City.

Mr. Byrne.

to be 22 ft. and the outer diameter 25 ft. This is shown by Fig. 1. The New Jersey terminal was to be on 12th Street, Jersey City, and the New York terminal on Canal Street, as shown by the plan and profile, Fig. 2. For lining, cast-iron was recommended, with a concrete facing. In 1910, this firm had prepared a plan for a two-level tunnel, Fig. 3. The lower level provided a roadway 16 ft. wide, with 9 ft. head-room; the upper roadway was to be 18 ft. 6 in. wide, with head-room of 10 ft. 6 in. at the curb and 12 ft. 3 in. at the center. On this upper level there were two footwalks for pedestrian traffic. It was contemplated that fast-moving automobiles would be placed on the lower level and all other traffic on the upper level.

The study for the arrangement of a roadway tunnel crossing beneath the Hudson River by Messrs. Jacobs and Davies, is based on the assumption that it is not probable that any large amount of pedestrian traffic would be likely to pass through such a tunnel, as the distance from shore to shore is about 1 mile at the region of Cortlandt Street, New York, and about $\frac{3}{4}$ mile at 59th Street. Hence, as much space as possible has been devoted to vehicular traffic, for which separate accommodation has been provided for "Fast" and "Slow", the fast being confined to the lower, and the slow to the upper, roadway.

Air ducts have been shown, to permit of mechanical ventilation.

Ample head-room is afforded for each type of traffic. For example: the highest slow-moving truck is about 10 ft. 3 in. high, and the head-room provided for vehicles of this class is 12 ft. 3 in.

The height of the highest limousine automobile is 7 ft. 10 in., and the head-room provided for fast vehicles is 9 ft. 0 in.

In July, 1916, the Public Service Corporation of New Jersey selected a Board of Engineers to report on the design and feasibility of a vehicular tunnel under the Hudson River. This Board consisted of W. H. Burr, Ralph Modjeski, and D. B. Moran, Members, Am. Soc. C. E., and, in January, 1917, it reported in favor of the construction of a tunnel by the trench method. The section in the river was to be of reinforced concrete resting on timber piles. It was to be, generally speaking, elliptical in section, the inner major axis being 28 ft. 4 in. and the outer axis 36 ft. 5 in. The over-all distance from top of piles to roof was 28 ft. 10 in. One roadway, 17 ft. wide with head-room of 13 ft., and two footwalks, one 5 ft. 6 in. and one 3 ft. 6 in. wide, were to be provided.

In September, 1917, Gen. George W. Goethals, U. S. A., M. Am. Soc. C. E., was retained by the Joint Commission to report on the best type of tunnel to be built. In November of that year he recommended the construction of a single tunnel with two levels, to be constructed by the shield method, and pre-cast concrete blocks were to be used in place of cast-iron with concrete lining. No change in location was recommended.

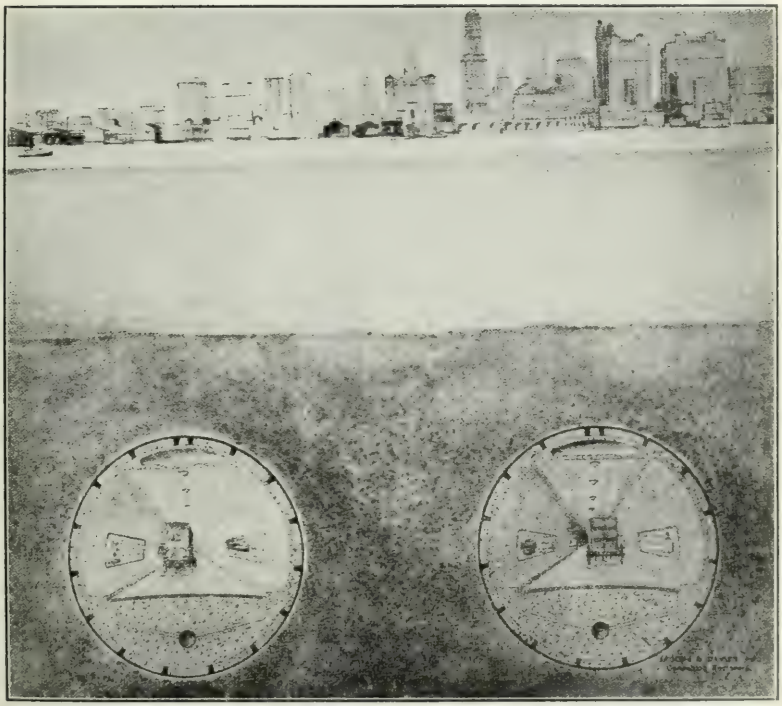
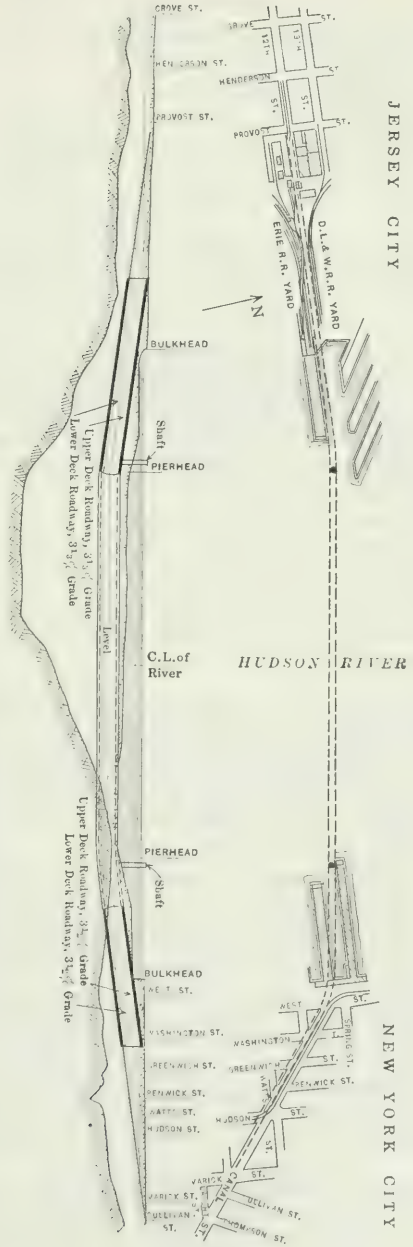


Fig 1.—Cross-Sections of One-Level, Two-Way Each, Vehicular Tunnels.

Mr. Byrne.



PLAN AND PROFILE OF PROPOSED NEW YORK AND NEW JERSEY VEHICULAR TUNNEL.

FIG. 2.

Mr. Byrne.

The following are the estimates of cost of the three tunnels:

1913—Jacobs and Davies, for two tubes, "pre-war estimate,"	$11 000 000
1917—Burr, Modjeski, Moran, one tube, "normal prices,"	6 899 000
1917—Gen. Goethals, one tube, "war prices,"	12 000 000

REASONS FOR THE CONSTRUCTION OF A VEHICULAR TUNNEL.

1.—Reduce vehicular congestion at the ferries.

2.—Prevent recurrence of possible coal and food famines, due to interruption of navigation on account of weather or labor troubles.

3.—Free part of river front now occupied by railroad companies for car-float and freight-lighter deliveries, by having freight delivered to New Jersey and hauled by trucks to New York, and *vice versa* for freight from New York. New Jersey has plenty of room for freight terminal expansion, but New York has practically none.

4.—Reduce cost of handling freight at New York by eliminating great trucking waste due to delay in receiving and delivering freight to railroad terminals. It will save the cost of rehandling at New York terminal. The expense of operating through the tunnel will more than offset lighterage and terminal charges in New York.

The construction of the tunnel has been recommended by all the substantial organizations of New York City.

The New York Assembly has passed a bill authorizing the New York Commission to enter into a contract with the State of New Jersey for its construction. This contract, to become effective, must be ratified by the United States Congress.

The Assembly bill carries an appropriation of $1 000 000, and, if it becomes a law, practically binds the State of New York to an expenditure not to exceed $6 000 000 as its share for the tunnel.

New Jersey has enacted legislation to provide a sum equal to that of New York.

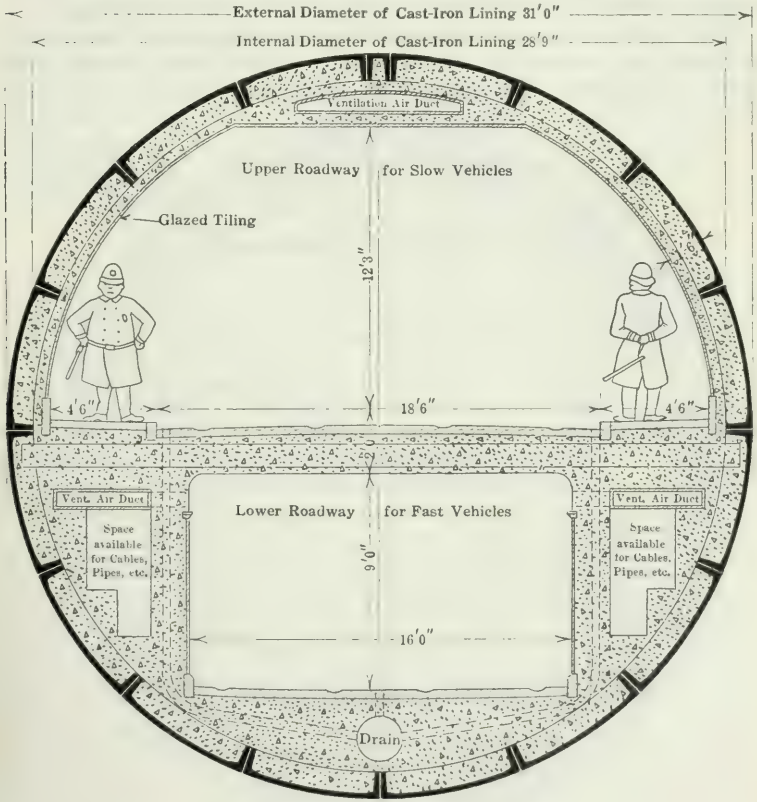
The estimated annual capacities of the three designs under consideration are:

Jacobs and Davies	5 000 000	vehicles.
Burr, Modjeski, Moran	15 417 600	"
Gen. Goethals	36 500 000	"

Messrs. Jacobs and Davies present no details as to what speed and spacing were assumed. Messrs. Burr, Modjeski, and Moran assumed 10 miles per hour and 60 ft. spacing, from center to center of vehicles.

In calculating the capacity of Gen. Goethals' plan, it was estimated that 100 000 vehicles in both directions could pass for 24 hours, as follows: Mr. Byrne.

	Speed, in miles per hour.	Distance, center to center.	Number of vehicles per hour.
Horse-drawn vehicles	3	50 ft.	300
Slow motor vehicles	9	60 "	800
Fast motor vehicles	15	80 "	990
Estimated hourly capacity			2 090
Total estimated capacity, in both directions, for 24 hours			100 320



ROADWAY TUNNEL ACROSS HUDSON RIVER.
STUDY FOR SUGGESTED ARRANGEMENT OF ROADWAYS, ETC. IN TUNNEL 31'0" EXTERNAL DIAMETER.
Nov. 18, 1910

FIG. 3.

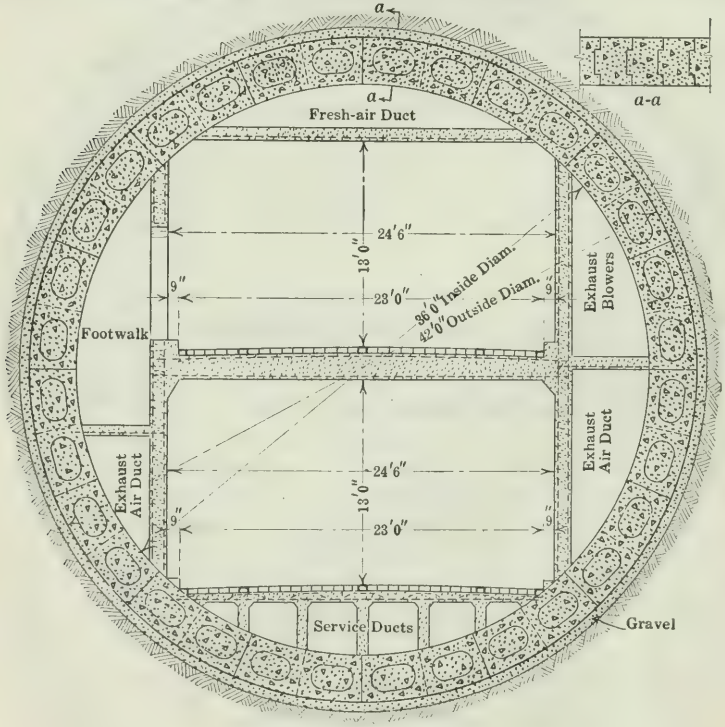
Mr. Byrne.

Applying Gen. Goethals' estimate to the design of Messrs. Burr, Modjeski, and Moran, and bearing in mind that the capacity for a single line of traffic must be controlled by the speed of the slowest moving vehicle, this tunnel would have a capacity of 14 000 vehicles in both directions per day, or 4 646 000 per year.

Applying Gen. Goethals' estimate to the plan of Messrs. Jacobs and Davies, it is estimated that 52 800 vehicles can be passed in both directions in 24 hours, or 19 272 000 annually.

The five ferries, at Cortlandt, Barclay, Chambers, Desbrosses and Christopher Streets, from which this tunnel would tend to take traffic, carry about 10 000 vehicles daily. It is estimated that the tunnel would attract at least 60% of this ferry traffic, or 6 000 vehicles daily.

The Delaware, Lackawanna and Western, Erie, and Pennsylvania Railroads carry across the Hudson daily 1 500 freight cars in each direction, a total of 3 000 cars daily. Assuming that these cars con-



TYPICAL SECTION
NEW YORK AND NEW JERSEY VEHICULAR TUNNEL
CROSS-SECTION OF A TWO-LEVEL, THREE-WAY EACH,
VEHICULAR TUNNEL
FIG. 4.

tain 20 tons each, 60 000 tons are carried daily. If one-half of this tonnage is hauled through this tunnel on trucks, with an average capacity of 4 tons each, it will require 7 500 trucks to do the work. Adding the traffic directed from the ferries, there results a daily total of 13 500 vehicles, or 4 500 000 annually (333⅓ days per year). Mr. Byrne.

Pleasure automobiles, which would be diverted from ferries north of Christopher Street, have not been taken into account in this calculation. Assuming 1 500 per day as a fair average, the daily total would be 15 000 vehicles, or practically the same number as Manhattan Bridge is now carrying.

This estimate is very conservative, and, if the increased traffic on the East River Bridges for the past few years is any criterion, we may confidently look forward to an increase in traffic of 100% in 10 years in this tunnel.

Table 1 is a statement of the vehicular traffic over the four East River Bridges, taken from the records of the Department of Plant and Structures, and is instructive and worthy of much consideration in determining the type of tunnel for the needs of the estimated traffic to be accommodated.

EAST RIVER BRIDGES. GENERAL STATISTICS.

	Opened.	Length.	Roadways.	Width.
Brooklyn Bridge.....	1883	6 016 ft.	2	16 ft. 9 in. each.
Williamsburg Bridge..	1903	7 308 "	2	20 ft. 0 in. "
Manhattan Bridge....	1909	6 855 "	1	35 ft. 0 in.
Queensboro' Bridge...	1909	7 449 "	1	52 ft. 0 in.

In designing any new bridge similar to these structures, no one would think of taking the roadway widths of the Brooklyn or Williamsburg Bridges as adequate for present-day requirements.

The Manhattan Bridge roadway is 35 ft. wide, sufficient to pass two lines of traffic in each direction. In order to care for the morning and evening rush-hour traffic on this bridge, horse-drawn vehicles are prohibited from crossing from Brooklyn in the evening period and from Manhattan in the morning period. During both these periods this traffic is diverted to Brooklyn Bridge, making it possible to accommodate three lines on Manhattan Bridge from Brooklyn in the morning rush and to Brooklyn in the evening rush.

The Police Department, in order to increase the capacity of Fifth Avenue, removed the isles of safety at 42d Street and at several streets north of that intersection. Formerly, only three lines of vehicles could be operated in each direction. By the removal of these obstructions four lines are now accommodated in the direction of the morning and evening rush periods, and two lines in the opposite direction. The Police Department reports that 18 800 vehicles pass 42d Street and Fifth Avenue in 10 hours.

Mr. Byrne.

TABLE 1.—Daily Vehicular Traffic

Bridge.	October 24th, 1912.			October 30th, 1913.			November 5th, 1914.		
	Horse-drawn.	Motor-drawn.	Total.	Horse-drawn.	Motor-drawn.	Total.	Horse-drawn.	Motor-drawn.	Total.
Brooklyn.....	3 114	799	3 913	2 909	1 305	4 214	2 643	1 340	3 983
Manhattan.....	3 074	1 749	4 823	3 723	2 323	6 046	4 005	3 611	7 616
Williamsburg.....	4 351	1 573	5 924	5 093	2 324	7 417	4 901	2 290	7 191
Queensboro'.....	1 354	2 290	3 644	2 036	4 538	6 574	2 002	5 205	7 207
Totals.....	11 893	6 411	18 304	13 761	10 590	24 251	13 551	12 446	25 997

Four East River Bridges:

1912, Total number of vehicles, 18 304; 1919, 46 305 vehicles. Increase, 150 per cent.

1912, 11 893 were horse-drawn; 1919, 9 750 were horse-drawn. Decrease, 18 per cent.

1912, 6 411 were motor-drawn; 1919, 36 555 were motor-drawn. Increase, 470 per cent.

1912, Manhattan Bridge, total vehicles, 4 823; in 1919, 14 613. Increase, 200 per cent.

Estimated tunnel traffic, first year after opening, 15 000 vehicles daily, with estimated increase of 100% in 10 years.

The four East River bridges have been the means of the discontinuance of seven ferries (Wall Street, Catharine Street, Roosevelt Street, Grand Street, Houston Street, 23d Street to Broadway, Brooklyn, and 92d Street) across the East River. Similar results, no doubt, will be obtained in the case of the ferries across the Hudson River when the vehicular tunnel is in operation.

The distance along the water-front from Pier 1 to 14th Street is 2½ miles, one-half of which is occupied by railroads for ferry and freight terminals. It is confidently estimated that at least ½ mile of this water-front could be released for other purposes if this tunnel were constructed.

The passage in the New York Assembly of the bill providing for the construction of a tunnel or tunnels, and appropriating funds to commence the work, has very forcibly brought the attention of the public to the work, and criticism has been directed against the design approved by Gen. Goethals. First, it was the matter of ventilation, one of the scientific papers claiming that it would be impossible to ventilate it artificially. This paper, during the past month, has reversed its former opinion.

Mr. Byrne.

COUNT OVER EAST RIVER BRIDGES.

OCTOBER 28TH, 1915.			DECEMBER 6TH, 1916.			OCTOBER 25TH, 1917.			MARCH 14TH, 1919.		
Horse-drawn.	Motor-drawn.	Total.	Horse-drawn.	Motor-drawn.	Total.	Horse-drawn.	Motor-drawn.	Total.	Horse-drawn.	Motor-drawn.	Total.
2 628	1 731	4 359	2 193	2 397	4 590	1 910	3 150	5 060	2 599	3 604	6 203
3 813	5 587	9 400	3 167	8 105	11 272	3 093	11 266	14 359	1 363	13 250	14 613
5 442	3 396	8 838	4 670	4 644	9 314	4 029	6 248	10 277	3 344	7 806	11 150
1 818	7 687	9 505	1 554	8 304	9 858	1 226	12 205	13 431	2 444	11 895	14 339
13 701	18 401	32 102	11 584	23 450	35 034	10 258	32 869	43 127	9 750	36 555	46 305

The size of the tunnel and the use of pre-cast concrete blocks in place of cast iron, are now being made subjects of controversy.

The Governor's Reconstruction Commission has invited criticism of the Goethals plan, and a Committee of Engineers has recommended that the type of tunnel for this location be decided by a Committee of Subaqueous Experts.

The points raised by the Reconstruction Commission are:

1.—Is the type of tunnel proposed possible of construction?

2.—After it is constructed, is it usable?

3.—Are the estimates of cost anywhere near correct—in other words, will the State find that it has entered upon a programme of much more expense than it has in sight at the present time?

The size of the tunnel is unusual; its diameter is 42 ft. 0 in., and the diameter of the largest tunnel yet built is 30 ft. 0 in. A special type of shield to drive the tunnel has been designed by J. F. O'Rourke, M. Am. Soc. C. E. This shield, which is divided horizontally into four sections, has multiple air chambers in its forward part, each ending 3 ft. in front of the chamber immediately above, thus enabling the excavation to be done in separate vertical faces. Gen. Goethals has approved this shield, and states that it will do the work of excavating the tunnel.

Concrete blocks have never been used as lining in a subaqueous tunnel. They have been used in a land tunnel in Montreal, driven by the shield method, and have been found satisfactory.

It is known that concrete blocks have been used for bulkhead wall construction for many years in New York City, and no fault has been found as to their durability.

The proposed tunnel will be 9 300 ft. long. The roof is 50 ft. below mean low water. This depth has been fixed in a permit issued by the War Department for a tunnel at this location. For 4 000 ft. the tunnel will be practically level. The grade of each of the approaches is

Mr. Byrne.

fixed at 3½ per cent. Practically similar grades exist on the East River Bridges. The lengths of roadway approaches and grades of these bridges are as follows:

Bridge	Approach	Length	Grade
Manhattan	Manhattan	2 900 ft.	3.10 per cent.
Williamsburg	Manhattan	2 500 "	3.00 " "
		1 500 "	3.5 " "
Queensboro'	Queens	3 600 "	3.4 " "
Brooklyn	Manhattan	1 562 "	3.25 " "

Ventilation experts have advised that the tunnel can be artificially ventilated. It has been stated that artificial ventilation is not necessary. This statement is based on the fact that two vehicular tunnels in London under the Thames are being used successfully without artificial ventilation, although at one of them the ventilation plant has been constructed and is ready for use if found necessary.

The London tunnels referred to are known as the Blackwall Tunnel and the Rotherhithe Tunnel. The Blackwall Tunnel, Fig. 5, is 6¼ miles below London Bridge. It is 6 200 ft. long and is lined with cast iron. The outer diameter of the tube is 27 ft. One roadway, 16 ft. wide, and two sidewalks, each 3 ft. 1½ in. wide, are provided. The minimum headroom is 15 ft. 6 in. The maximum grade is 1 in 36 (2.78%). The tunnel was opened to traffic in 1897. The contract price was £870 000; the cost of property, sites for rehousing, and miscellaneous, amounted to £536 000, a total cost of £1 406 000. The 1917 Report of the Public Service Corporation of New Jersey states that the annual traffic for 1913 was 772 940 vehicles and 3 742 241 passengers. An actual count during a period of 13 hours, in January, 1916, showed that 2 000 vehicles and 7 473 passengers used this tunnel, the greatest number of vehicles for any hour being 219.

The Rotherhithe Tunnel, Fig. 6, is 2¼ miles below London Bridge. It is generally similar in design to the Blackwall Tunnel, with the exception that the outer diameter is 30 ft., making it the largest subaqueous tunnel ever built. It is 6 826 ft. long, and has a roadway 16 ft. wide and two footwalks, each 4 ft. 8½ in. wide. The maximum grade is 1 in 37 (2.7%). The length of the river portion, between shafts, is 1 535 ft. The tunnel was opened to traffic in 1908. The contract price was £1 088 484; the cost of property, sites for rehousing, and miscellaneous, amounted to £911 516, a total cost of £2 000 000.

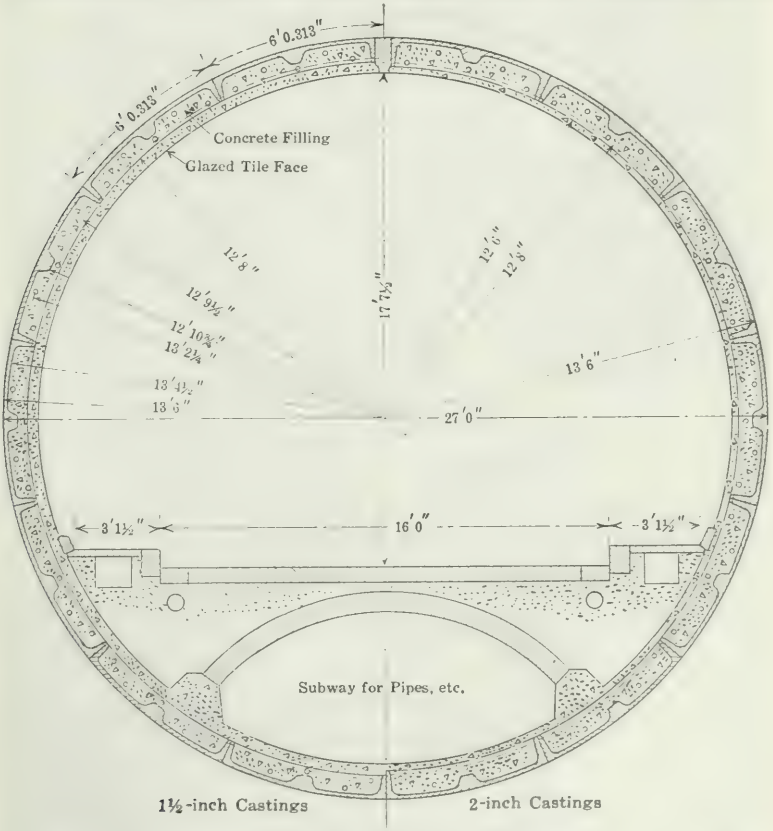
The 1917 Report of the Public Service Commission of New Jersey states that the annual traffic for this tunnel for 1913 was 937 946 vehicles and 3 360 067 passengers. From a count made in 1914, for a period of 12 hours, 8.00 A. M. to 8.00 P. M., 2 040 vehicles used this tunnel.

The heaviest traffic for any one hour, from the foregoing records of these two tunnels, was 219 vehicles for the Blackwall Tunnel.

The speaker's estimate of traffic through the proposed tunnel, of 15 000 vehicles per day, would mean a maximum of 1 300 vehicles per hour of heaviest traffic. This estimate is made from the count taken at the Manhattan Bridge on March 14th, 1919, on which day there was a total of 14 613 vehicles recorded as using the bridge for 24 hours, the heaviest hour traffic being 1 166 vehicles. Mr. Byrne.

At a hearing before the Reconstruction Commission, Mr. O'Rourke submitted a proposal to build the tunnel complete ready for use for the sum of $11 500 000.

In answer to the criticism raised at the hearing, that the patented shield, concrete blocks, and gravel backing gave him exclusiveness in

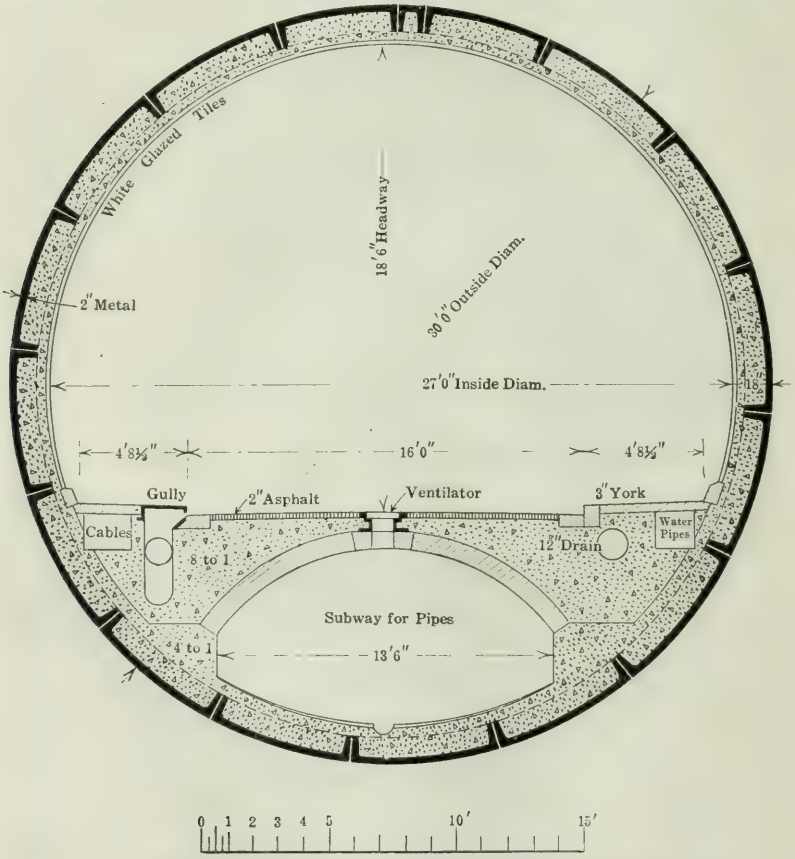


BLACKWALL TUNNEL, LONDON
CROSS-SECTION OF TUNNEL, SHOWING TWO PATTERNS OF CAST-IRON LINING USED.

TYPICAL CROSS-SECTION OF A
LONDON TWO-WAY VEHICULAR TUNNEL.
FIG. 5.

Mr. Byrne. bidding for the contract, he proposed to refer his interest in the matter to the Chamber of Commerce, with full power to fix his compensation for giving license to any contractor to use all the patents for the driving of this tunnel. This compensation was not expected to exceed one-quarter of the saving made by their use.

From an experience of many years with traffic on long-span bridges, there is no doubt in the speaker's mind that narrow roadways are not to be recommended for present-day traffic, as they are sources of congestion and blockade, particularly during the heaviest traffic periods. The failure of one vehicle to maintain its position at these times has been frequently the cause of the entire interruption of the vehicular



TRANSVERSE SECTION OF THE NEW ROTHERHITHE TUNNEL.
TYPICAL CROSS-SECTION OF A
LONDON TWO-WAY VEHICULAR TUNNEL.
FIG. 6.

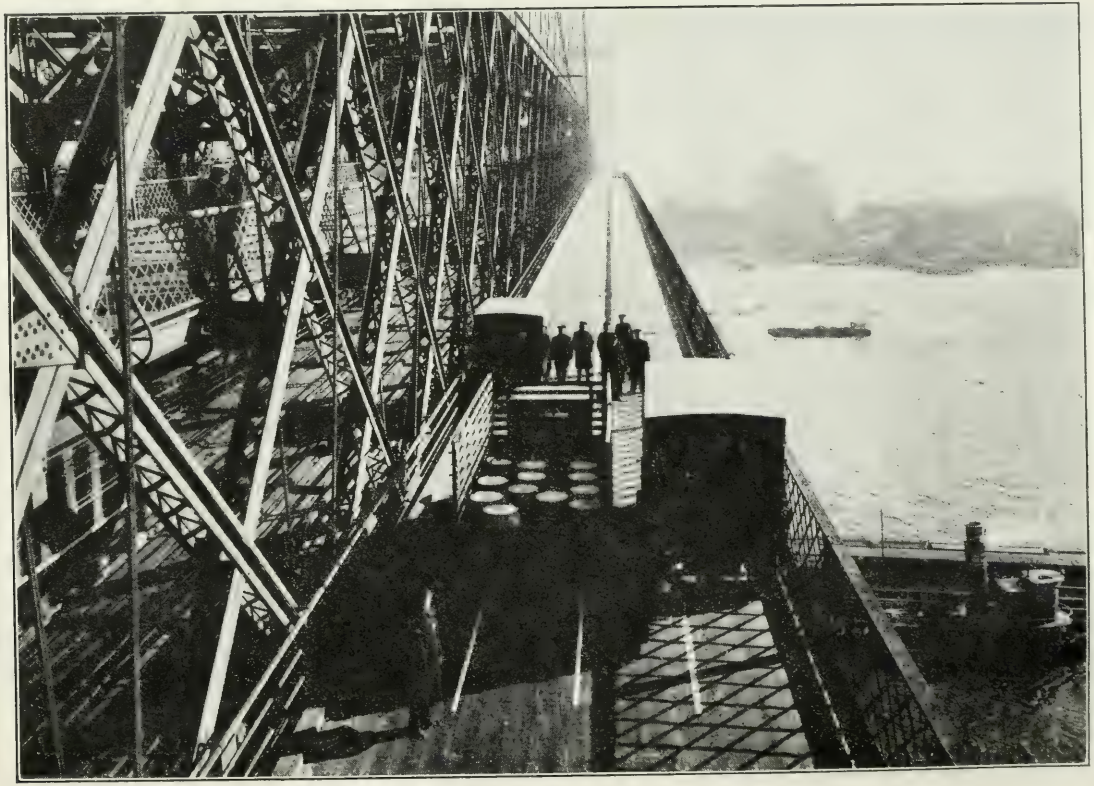


FIG. 7.—SHOWING HOW AN ADDITIONAL 3 FEET, TO A 20-FOOT ROADWAY, WOULD PERMIT OF THREE-WAY TRAVEL.

traffic across bridges which have narrow roadways. The Brooklyn and Williamsburg Bridges are examples of this narrow roadway error, but the roadways of the Manhattan and Queensboro' Bridges are free from congestion from this cause. Fig. 7 shows that the addition of 3 ft. to a 20-ft. roadway would permit of 3-way travel. Mr. Byrne.

It would require four 2-line vehicle roadway tunnels to equal the capacity of the tunnel approved by Gen. Goethals. The cost of these would be $22 000 000, by pre-war estimates, as compared with $12 000 000, by present-day prices, for the Goethals design.

Therefore, from a cost and efficiency point of view, the problem seems to be easy of solution.

JOHN F. O'ROURKE,* M. AM. SOC. C. E.—Mr. Byrne's description of the general conditions of the tunnel traffic question—about which he is probably as well informed as anybody in the world—and also what he has shown of the different types of tunnels, including that in which the speaker is interested, will probably leave little to be said on the subject of highway traffic, and so this discussion will be confined to certain novel features of the proposed Hudson River Vehicular Tunnel and some of the new methods involved. Mr. O'Rourke.

It is properly said that that tunnel is a large one, and that there has been no other like it in some of its features, but, in general, it is similar to the other Hudson River Tunnels, and in its vital features is exactly the same. The shield, also, is like other shields in all essentials, and, in addition, possesses two simple improvements which would have been used on former shields if they had been known.

Certain patents have been issued to various parties for methods of dividing the face of the shield into different levels, so as to avoid the unbalanced pressure caused by a height equal to the whole diameter, which exists where there is one chamber from top to bottom at the front of the shield. The first of these patents, for subdividing the front of the shield, was issued to Brunel in 1818, but it had nothing to do with compressed air, because that was before its use had been introduced in tunneling. The first patents for multiple air-pressure shields were issued about 1888. In 1891 the Pearsons, of S. Pearson and Son, of London, were granted a patent on a multiple air-pressure shield, the drawings of which show four levels, the same as the shield now under discussion.

At about the same time, the closing of the opening between the tunnel lining and the tail of the shield, in order to prevent the escape of air or the inflow of water, was attempted by Sir E. W. Moir, M. Am. Soc. C. E., of that company. He obtained two patents on the subject. D. L. Hough, M. Am. Soc. C. E., also obtained patents on means to close that opening.

* New York City.

Mr. O'Rourke.

The speaker has no knowledge that any of the tail-closing devices were ever tried, but the multiple air chambers were, and were not a success, because the faces of the excavation were all in one vertical plane. When the excavation had been carried sufficiently far downward, the air crept around the face of the floors from one level to another until the same pressure existed from bottom to top in all the chambers. In the new shield, as each chamber is projected beyond the one above, this cannot happen, because the air passes into the ground, where it expands as it rises, passing each chamber at less than the pressure in that chamber.

If there is anything that the speaker's experience with compressed air has taught him, it is that as soon as air escapes into the ground it expands and rises, for, no matter what its pressure was when it got out, it instantly takes the pressure of the ground-water it is in. The several chambers have the same air pressure as the ground-water at the level of their floor, and thus have a higher pressure than that in the ground in front of them, and, consequently, the escaping air cannot hold enough pressure to enter any chamber it is passing. If, for any reason, it moved backward as well as upward, across the step between the vertical planes of the excavation, the ground friction would cause an additional reduction in its pressure, further preventing it from entering the chamber above. Therefore, nothing is clearer than the fact that air escaping into the ground from one chamber cannot enter another chamber in which the air pressure is greater than that of the ground-water in front of it.

It has been stated that perhaps a boulder might be met at the front of a floor when shoving the shield, a contingency which is most likely. In pulling in or blasting that boulder, a hole is made of the same size as the boulder. The experts say: "What is to prevent air in large volume from going through this hole and entering the chamber behind?" The answer is that a large volume of air could not be contained in a hole the size of a boulder, and that the boulder must have been at least the width of the step in front of the chamber. All these things relate only to the work on shore, where sand and gravel are found. Under the Hudson River there is only silt, and the upper levels of the shield would never be opened to take in any of that. The effective cover could be counted on as from the top of the opening through which silt is being admitted into the shield, to the bed of the river or the top of the clay blanket, as the case may be.

As already stated, the efforts heretofore made to close the opening, between the tail of the shield and the tunnel lining within it, reached the point of taking out patents for devices. Stuffing the opening at the top with cement bags, or blocking it with steel plates against the end of the last ring, are about all that has been done to impede the escaping air. The bags were blown out at the next shove, and the

steel plates had to come down in order to erect the next ring, so that no real continuous checking of escaping air was accomplished. As the back of the shield under the lining was inaccessible, nothing was ever done to stop the water from entering at the bottom, except to raise the air pressure while the bottom plates were being set, whenever the water gave trouble. In order that the multiple air-pressure chambers at the front of the shield might have practical value, it was essential that this opening should be closed continuously against the escape of air or the inflow of water. Mr. O'Rourke.

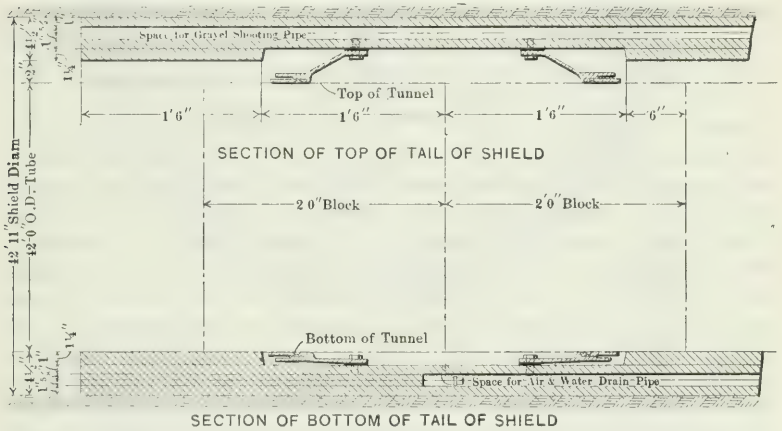


FIG. 8.

Fig. 8 shows how this is to be done by the simple expedient of providing a recess in the tail of the shield by two 1¼-in. steel-plate rings having a space between them in which two flap-valves, extending around the whole circumference, are fastened to the tail. The back one faces backward, and the water seeking to enter presses it down on the lining, making a water-tight seal, like the foot-valve of a pump. The front one faces forward, and the air pressure forces the flap down on the lining, making an air-tight seal, like a valve in an air compressor.

In this way the water is excluded and the air is contained, irrespective of the water or air pressure, so that, in connection with the multiple air-chambers, the excavation may be done without much unbalanced pressure under greatly reduced cover, and the tunnel itself can be given such pressure as may be most desirable, independent of the immersion.

It is likely that some lubricant may be used on the lining, to facilitate the slipping of the flap-valves during the shoves, and provision will also be made to pump back into the tunnel any water or

Mr. O'Rourke.

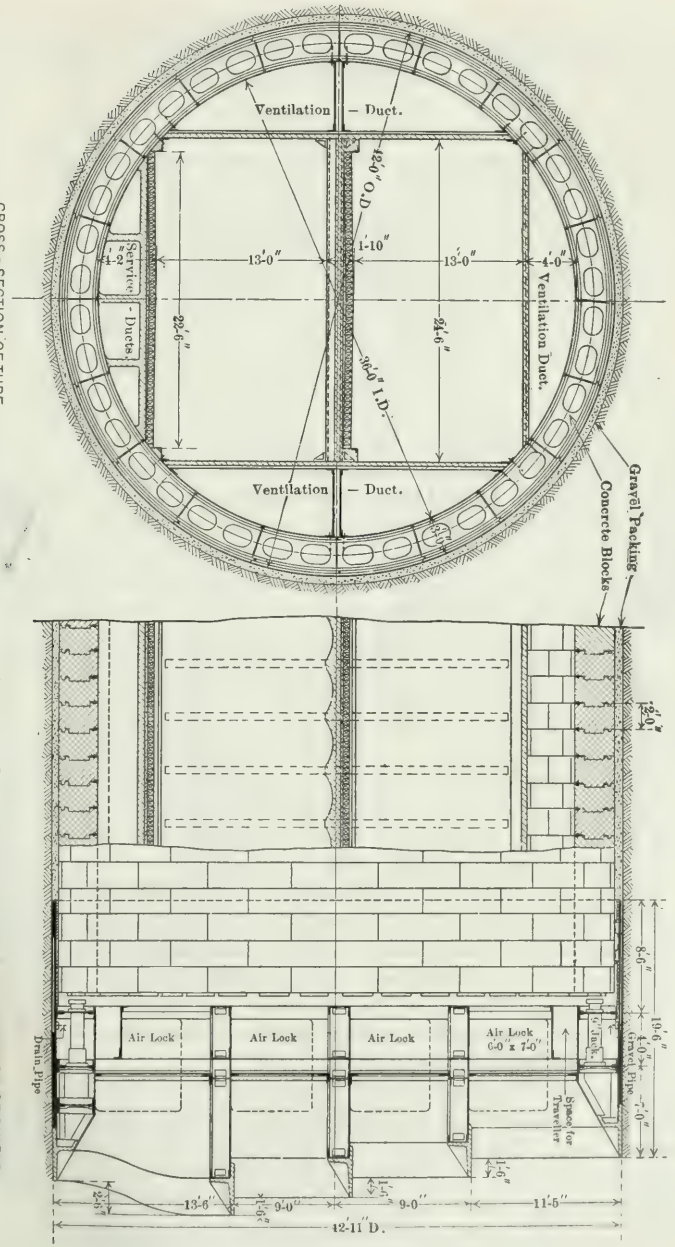
air that may accumulate between the valves and interfere with their operation. The valves will be strongly constructed, and should last through the distance the shield has to go, but, if necessary, they can be readily repaired or replaced.

Another matter which should be mentioned is the information obtainable in regard to the Canal-12th Street Line. It is obvious that that location must be right, because everybody who has projected a down-town tunnel has selected it. It was chosen first by Messrs. Jacobs and Davies. It was afterward adopted by the Commission of Engineers for the Public Service Corporation of New Jersey, and was also chosen for this latest tunnel. It is to be borne in mind that the Commission of Engineers, appointed by the Public Service Corporation to investigate the Hudson Vehicular Tunnel question, was given *carte blanche* as to the information it was to obtain, and also the extent to which its expenses might go. The three men on this Commission are among the most prominent bridge and foundation engineers in the country, and have had great experience in obtaining data for plans. The work of exploration required for foundations for bridges or buildings is necessarily much more extensive than any exploration for a shield-driven tunnel. In one case the work is from the top down, and it is important to know the material in which the excavation will be made and the kind of material at the bottom. In the case of shield tunnels, what is above the tunnel is of little importance, except as to whether it will or will not answer for cover. It is necessary that the ground under the tunnel be sufficiently compact to carry the load, and this can be done, even by silt, as experience shows, probably because the structure weighs less than the ground it displaces. As the shield can be driven through any kind of material, a few good borings answer all practical requirements, particularly in the Hudson River, where the shores and river bed have been tunneled so often. The same parties who were employed on the speaker's Pennsylvania Tunnels were engaged by the Commission, and made thirty-eight core borings into the rock from end to end of the tunnel site, even where the rock was 200 ft. below the grade. Therefore, the rock profile is very well known.

In addition to these borings, forty-two test piles were driven in the river bed. As the bed of the river consists of silt above the rock, with a foot or two of gravel and sand lying on the rock, the Commission is justified in the statement made in its report that it obtained the first and only complete rock profile under any of the rivers about New York City.

Fig. 9 shows the proposed interlocking concrete block tunnel, and the details of the blocks are given on Fig. 10. The projections are $\frac{1}{4}$ in. longer than the depth of the recesses, so that the blocks do not come in contact, except at the bottom of the recesses; that is to say,

Mr. O'Rourke.



CROSS - SECTION OF TUBE

LONGITUDINAL SECTION AND VIEW OF TUBE; SECTION OF SHIELD

FIG. 9.

Mr. O'Rourke.

the pressure caused by shoving the shield is transmitted through the lines of projections and recesses, so that spawling is avoided, and the other parts of the joints are free and accessible for water-proofing, caulking, and grouting.

A key-way is moulded at the top of the joints, on one side and one end of each block, in which is put a mastic gasket, opposing which are projections from the opposite side and end, which compresses the mastic, thus completely filling all joints at the top.

At the bottom of the joints a caulking seam will be moulded, and filled with a well driven caulking of oakum and tar, as a final line of defense against the entrance of any water that might pass the mastic and grout portions of the joints. Three blocks will be cast at a time, so that where the key-block shows at the top, requiring three special blocks, those three would be made in one operation. The parts of the joints inside the key-way and caulking seam, except a narrow smooth band for an exact ¼-in. joint in which spacers will be placed while the blocks are being erected, will be left purposely rough.

The size of the block, measured from the bottom of the recess to the face of the projection, must be uniform throughout, and all the moulds will have machined faces at these points, maintained at exact distances apart. There ought to be no more difference in the blocks, as to projections and recesses, than in fine machine work, because concrete fits its mould, and the moulds would be made with extra care at those points. Therefore, the blocks can be relied on to maintain contact throughout the line of projections and recesses. The jacks never let go of the blocks, except that when an adjoining block is being set, one jack is released on each side of a joint, after which the pressure is immediately restored. This means that the concrete block rings are always under compression. Assuming one-fourth of the jack power to be used for compression between shoves, this would equal about 50 lb. per sq. in. of ring. Concrete weighs 1 lb. per sq. in. for each foot of height, so that the rings are always under a compression equal to the weight of a section of tunnel, 50 ft. long, turned up on end; thus the rings are like courses in a pier, 50 ft. below its top, which are held in place by that much superimposed weight. Therefore the concrete throughout the tunnel is given an initial compression which renders tension members unnecessary, as no longitudinal tension can exist.

It is proposed to use gravel packing around the tunnel lining throughout. This gravel is blown simultaneously through a number of rubber hose pipes inserted in passages in the tail of the shield. The gravel will be blown from many tanks into the space made by the shield outside the lining as it moves forward, completely filling that space. The gravel is to be composed of an equal mixture of coarse and fine grits, which are about the size of buckshot and birdshot.

Mr. O'Rourke

The effect of the impact of the gravel on itself is to crush it into a hard mass, cemented like embryo sandstone by the powder into which some of it is reduced. In building inter-track stations, there has been occasion to take out portions of tunnel lining, where gravel packing had been used, and the lining seemed to be encased in sandstone. The Public Service Commission engineers took photographs showing cracks in the ground into which the crushed gravel ran, the same as grout would, and it was equally hard, for all practical purposes. In connection with the Hudson River silt, it is certain that gravel packing will hold the ground undisturbed and solidified where it surrounds the tunnel, so that its bearing power will be greatly improved.

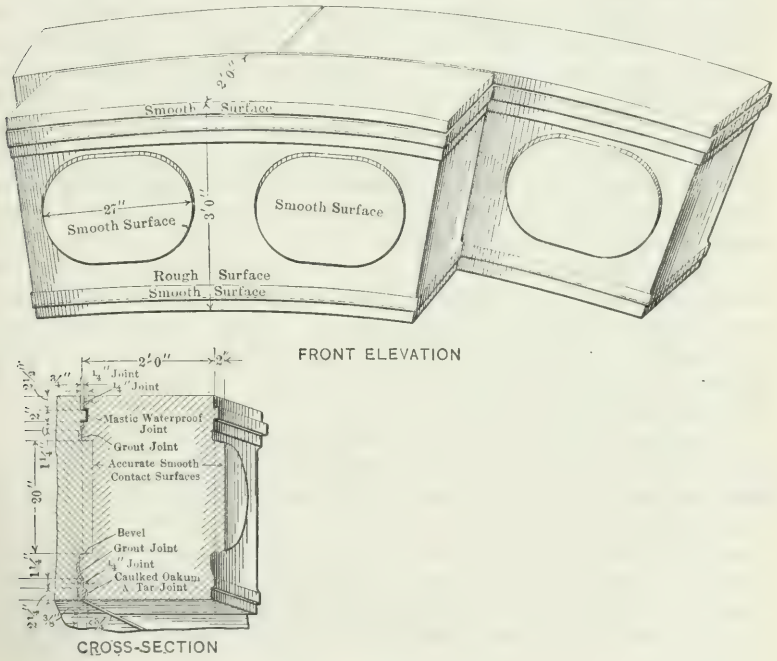


FIG. 10.

Some question has been raised as to the use of gravel packing in silt, through which the propelling air could not escape. In places where the ground is impervious, the shield provides means for returning the air that carries the gravel back into the tunnel through the same passageway that contains the hose. In a tunnel in Detroit, where bad settlements were occurring, gravel packing is now being used successfully in clay.

Mr. O'Rourke.

The experience during the construction of the Pennsylvania Tunnels was that there were serious irregularities of pressure in the silt against the sides of the lining for about a year after the tunnels were driven. Shores and tie-rods were used throughout to maintain the rings in circular form, and these had to be retained for more than a year, after which the ground had settled uniformly around the tunnels, and the cast-iron rings retained their form unaided while the concrete lining was being placed. This would indicate that, if gravel packing had been used to prevent the disturbance of the silt, there would have been no trouble from unequal pressures at any time. It also proves that the alleged differences of pressure in undisturbed ground around the circumference have no distorting effects. A test was made in the Willoughby Street Tunnel by slacking off the turnbuckles. This was done without causing the slightest spreading of the sides of the tunnel, so that it is safe to say that the gravel packing enables the ground to hold the tunnel and the tunnel to hold the ground, in silt as well as in any other material; and that this large tunnel can be built, if gravel packing is used, without any deformation of the concrete rings or disturbance of the equilibrium of the ground. Moreover, the 36-in. interlocked concrete block rings are very much stronger than any cast-iron rings that could be built within reasonable limits of cost, and there is no question that where the weaker ring held its form assisted only by gravel packing, the stronger one will also, with the same assistance. Besides, these strong rings get further strengthening from the steel columns and floor, securely anchored to the lining so as to take tension or compression at six different parts of the circumference.

The lowest point in the Pennsylvania Railroad Tunnels was 97 ft., and the air pressure was about 25 lb. per sq. in. from one side of the river to the other, that is, much less pressure was required than the immersion would indicate. That answers the question, asked by Paul G. Brown, M. Am. Soc. C. E., as to the men in the bottom chamber working for short shifts while the men in the other chambers would be idle. As a matter of fact, this shield and this method do not call for any greater immersion than the Pennsylvania Railroad Tunnels.

The rock at the New York end of the tunnel would be taken out in a shaft, and not under compressed air.

In reference to the ventilation of the tunnel, an article * appeared a short time ago which was not meant to apply to this tunnel, as it had not been proposed when the article was written. In attempting to describe it, the writer of that article alluded to Mr. Davies' tunnel and to the tunnel of the Public Service Corporation. He assumed 52 sq. ft. as the area through which were to be diluted the fumes of

* *Scientific American.*

about 150 automobiles, averaging 15 h. p. each, and all supposed to be consuming gasoline at the highest rate. He then calculated the velocity and power required by using Kent's formula for ventilating mines. He assumed that the rough surface of the mine passages and the smooth surface of a ventilating duct have the same modulus of friction. As it is 0.002 in one case and 0.007 in the other, it will be seen that there is some difference. That article, however, is used to-day against this tunnel which has 250 sq. ft. of duct area, not to mention more than 600 sq. ft. of roadway which will also be used. Mr. O'Rourke.

The well-known law in regard to the movement of air is that the power required to move it varies with the cube of the volume, or inversely as the cube of the area of cross-section of the tube. If there were 52 sq. ft., 48 000 h. p. would be required, costing $2 600 000 a year. A duct having 104 sq. ft. would require one-eighth as much, or 6 000 h. p. Doubling that area again would give one-eighth of one-eighth of 48 000, or about 800 h. p. The actual duct area of this new tunnel would require about 500 h. p.

Ventilating apparatus was provided for the Blackwall Tunnel, but was never used, and no provision for ventilation was made in the next Thames tunnel. There are half a dozen other vehicular tunnels, none of which has induced ventilation, and where even pedestrians go without danger or inconvenience.

Mr. Parmley has discussed the subject of jacks and the slope that the earth takes around a tunnel, and how much more jack power will be needed. The speaker is familiar with nearly every tunnel that has been built in the vicinity of New York City, and has had to do with a great many kinds, both as engineer and contractor. There is one rule in regard to jacks and resistance to jacks in shields that is as true as that in regard to the weights put on a caisson in order to sink it, and that is, it is a matter of the area of rubbed surface. The diameter or dimensions do not matter. It is the rubbed surface that counts.

There is a rule, and it is more for construction reasons than anything else, that jacks in a shield are placed about 3 ft. apart, so that, if the diameter of the tunnel is known, the number of jacks required will be the same as the diameter, in feet. If a tunnel is to be driven under Brooklyn Heights, where there is 120 ft. of cover, and where the weight on top is several times as great as that on top of this tunnel through the Hudson silt, that rule could be followed as successfully as the speaker has followed it, and no calculations will satisfy him that the jacks he used did not push those shields, because they did.

Then, again, the question of the design of the blocks and the load that is forced on the blocks by jacks, was fully settled at Montreal, where there is a cover of more than 60 ft. above the tunnel, at least 40 ft. of which is that jelly-like substance known as "bull's liver".

Mr. O'Rourke. The jacks used in the shields there were loaned by the O'Rourke Company to the contractors, Mackenzie, Mann and Company. S. P. Brown, M. Am. Soc. C. E., the Chief Engineer of the tunnel adopted these blocks without any precedent to guide him, and he and his principals are greatly pleased with the results. The speaker was familiar with the work while it was going on, and knows whereof he speaks.

At a recent hearing, in regard to the Hudson River Tunnel, before the Reconstruction Commission of the State of New York, Mr. Brown declared that, although the weight was so great that it crushed the footings of the shield, consisting of three 12-in. I-beams bolted together, with concrete in between, bent the center columns, and sometimes sheared the rivets of the channel-beams at the top of the columns, in no case was a concrete block even deformed. Further than that, with all this weight above, the jacks, approximately 3 ft. apart, pushed the shield, exerting 150 tons when the water pressure was 5 000 lb., and there was not a single case in which a projection on a concrete block was crushed. In other words, this tunnel with interlocking concrete blocks was driven most successfully under difficulties fourfold as great as those that will be encountered by the tunnel in the Hudson River silt.

The use of concrete blocks saves both time and money; nevertheless, the speaker's company is prepared to use either cast-iron or concrete lining.

Mr. Brown. Paul G. Brown,* M. Am. Soc. C. E.—Only two weeks ago the Governor of New York State requested Engineering Council to appoint a committee of five engineers to advise the Reconstruction Commission of the State of New York in regard to proposed public works projects. To act on that committee two engineers of this Society were chosen. One of the first things the Governor asked the Reconstruction Commission to do was to report on the vehicular tunnel under the North River. It happened that only one member of the Engineering Committee had had tunnel experience, and as the Committee believed that it ought not to pass on such an important work, involving so many technical details, it recommended that the Governor refer the matter to a paid commission of properly qualified engineers.

Some months ago this Society, thinking that it was not performing its full functions in the affairs of the State and the Nation, appointed a Development Committee. This Committee has not yet reported, but, as it has seemed to be the desire that this Society should fill a large place in the community, the members now have a chance to assist the Governor of the State and perform that larger function, for, if those who are familiar with this kind of a proposition will say what they think about it, they will do a very good work for the State, and will

* New York City.

help the Governor, as he is very anxious to get all kinds of public improvements going. Mr. Brown.

There are many members of this Society who are capable of discussing such a project. Mr. M. L. Quinn has had much experience in tunnel construction, as well as Messrs. Davies, Forgie, Lavis, Mason, Haggard, Snyder, and Shailer; and there may be many others. If they will say what they think about this tunnel, the Committee and the Society can get behind their opinions without any question.

The Committee has been asked a number of questions, a few which are as follows:

Can such a tunnel be ventilated?

Can a shield, 43 ft. in diameter, be constructed of sufficient strength, without prohibitive weight and expense?

Would pre-moulded concrete blocks have sufficient strength to withstand the thrust of such a shield?

Could pre-moulded concrete blocks of the design suggested be made water-tight?

Would such blocks have sufficient strength to support the tunnel structure?

Is it possible to maintain compressed air at several stages of pressure in the face of a shield?

With four sets of air locks in the shield, would the concentration of so much effort at this point prove a serious obstacle?

Can a shield of such size and strength be kept to line and grade?

It is a well-known fact that a shield, if it cannot move ahead, will move downward; and, with the lower part of such a shield projecting so far beyond the upper, if it did settle, could it ever be brought up to grade?

This shield is 93 ft. below mean low tide, or 20 ft. lower than would be necessary with a smaller tunnel with correspondingly lower pressure. In the bottom chamber, through the rock sections, the "sand hogs" would only work short hours. The shield could move no faster than the bottom chamber. Would this make the progress so slow that the cost would be prohibitive?

Can any system of concrete blocks be designed for this tunnel that would be a proper substitute for the usual cast-iron lining?

Would the congestion of traffic at the terminals, due to the concentration into one tube, prohibit that type of construction?

Would two single tubes, if carrying two lines of traffic in each direction, be sufficient for the present traffic needs?

What units of cost are used in making the estimate of $11 500 000 for this tunnel?

The speaker hopes that this discussion will bring out many facts concerning this project.

Mr. Davies.

J. V. Davies,* M. Am. Soc. C. E.—The speaker is deeply interested in Mr. Byrne's presentation of this proposition, particularly with reference to the traffic. Information in respect to the actual volume of traffic to be expected in connection with these proposed highway tunnels has always been more or less deficient, and Mr. Byrne's statement of that feature is decidedly illuminating.

The speaker admires Mr. O'Rourke's presentation, and the ingenuity with which he solves all the various problems in connection with his tunnel. As Mr. Byrne, in his opening discussion, has mentioned the speaker's name, it would be well to clear up a little history in that regard.

A week ago, the speaker attended a meeting of the Reconstruction Committee, and stated then that he had been connected with the Interstate Commissions more or less, for some years. About 1910 the speaker's firm made a study of this proposition, and, as tunnel engineers, came into connection with the New Jersey Commission, in the first instance, and, later, with the New York Commission, in working out this vehicular transportation proposition.

In 1913 his firm reported to the Commission on this subject, and advised that two plans were feasible; one was a tunnel of large size, approximately 30 ft. in diameter, which could be built in pairs, one for east-bound and one for west-bound traffic. Each of these could be further divided, for slow vehicles on one level and for fast vehicles on a second level, and thereby give great capacity for traffic.

The other proposition was for smaller tunnels, of the general size of those built by the Pennsylvania Railroad; and, at the request of the Commissions, that one single and definite plan should be reported on to the Legislatures, his firm prepared the report of the Commissions, which was forwarded in 1913 to the Legislatures of the States of New York and New Jersey, recommending a pair of tunnels of the smaller size.

Now, the point of this was very simple. The members of the speaker's firm had no information, other than their own personal knowledge, of the conditions north and south of the location that had been laid out for this vehicular tunnel, and were not prepared to make any wild gamble as to what might be done, but preferred to stand by what they knew could be done; and, further, the two Commissions had no money available with which to make investigations of any sort.

In 1914, George D. Snyder, M. Am. Soc. C. E., presented this subject of subaqueous vehicular tunnels, in a paper† which contains most of the structural information presented by Mr. Byrne. That paper treats of all subaqueous vehicular tunnels, and covers Mr. O'Rourke's proposition, as well as the various plans of open-trench tunnels.

* New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 252.

Mr. Davies.

Now, there is one thought that should be brought out clearly, and that is, that there never has been at any time any assembling of these various propositions, so as to compare them on the same basis of costs. Mr. O'Rourke's plan for a very large tunnel may be quite feasible, but it would have to be considered in detail before the speaker would feel equal or capable of saying whether or not it was desirable. These plans must be brought to a sane basis for comparison.

Mr. Byrne states that the tunnels, to give a capacity equal to Mr. O'Rourke's design, would require four of the type which the speaker had recommended to the Commission, and would cost $24 000 000, whereas Mr. O'Rourke's design would cost $11 500 000. There is evidently something radically wrong in such a comparison. This thought should be brought up squarely. The displacement and consequent excavation of a 43-ft. tunnel, which represents the shield that Mr. O'Rourke has designed, represents 53 cu. yd. per running foot. A pair of 24-ft. tunnels represents only 33 cu. yd. per running foot. There is 60% more excavation per running foot in the large tunnel than in the two of smaller size.

Of the cost of construction of a tunnel, more than 50% is represented by the labor and other costs expended in producing the hole, that is to say, the excavation and contingent work to prepare for the insertion of the lining; and probably not more than 20% of the total cost is in the iron lining itself. This fact is worthy of careful consideration.

Then, another point: The builder of this large tunnel must consider that the roof grade is approximately at the same roof grade as the smaller tunnels, that is, any excess in diameter is at the bottom. The intersection with the rock reefs on each side comes at earlier points at the two ends, and the most expensive part of the tunnel driving is from the moment the rock reefs are touched and work has to be done in a rock bottom with a soft ground top; thus, the increase in the cost of the excavation comes sooner and is in larger amount.

There are approximately only 3 000 ft. between the rock reefs on each side. On the New Jersey side, above the rocks there is sand, which is as bad as rock, inasmuch as every bit of it has to be excavated. The shield cannot be shoved into it an inch until it has been excavated. That proposition has to be considered.

Another point, with respect to New Jersey, is the fact that any new tunnel must be located so that it will be above the plane of the tunnels of the Hudson and Manhattan Railway, the tubes of which at the location of intersection, are superimposed, lying one on top of the other, leaving a comparatively shallow depth between the surface of the yards of the Erie and Lackawanna Railroads and the roof of the tunnels below, so that whatever construction, of whatever type, must go above those tunnels at that point of intersection.

Mr. Davies. Therefore, although this discussion should, and undoubtedly will, draw out a lot of very valuable information, it is not thought that any one will arrive at a very definite conclusion; and, further, all these plans should receive very serious and careful consideration on their merits, at some time or other, before any definite, final adoption of any one of them is made.

Mr. Forgie. JAMES FORGIE,* M. AM. SOC. C. E.—The speaker is surprised to find that this public highway tunnel discussion pertains chiefly to a shield-driven tunnel of definite design, having an external diameter of 42 ft., and to be lined with pre-cast concrete blocks.

The economic necessity of a highway tunnel under the Hudson is quite obvious, and is only second to the pressing necessity for a railroad freight tunnel connection, though it is of little consequence which is built first. In 1892-96, in London, they built a vehicular tunnel, in which the speaker obtained his baptism of compressed air, the Blackwall Tunnel. This tunnel was so successful that another was built, the Rotherhithe Tunnel, completed in 1908. London's industrialism can distribute itself in almost any direction, and is not confined, as is that of Manhattan; so that, for New York City there must be still greater reason for a vehicular tunnel to connect it with the mainland. New York City is a clearing house for the continent, and in that capacity has a very great, pressing, and all-sufficient duty to discharge.

The speaker feels considerable diffidence in saying anything on the matter of design and method of construction of this tunnel; but is attracted by Mr. O'Rourke's proposed cross-section which is to accommodate two superposed roadways, each with three lines of traffic in one direction. Of course, a one-way road of three lines of traffic has advantages over one with two lines of traffic, but, to realize it in one tunnel, in the way suggested, may or may not be economical.

There are many things that require serious study and consideration in connection with the construction of a tunnel of normal size, and each tunnel project, even under the same waterway, becomes an individual problem. Much as the speaker would like to give his unqualified opinion, he does not think it wise to do so, on this occasion, on this large tunnel, of 42 ft. external diameter, lined with concrete blocks, and proposed to be constructed with a multi-air-pressure shield. Therefore, he wishes it to be understood that anything he may say should be considered as general, but definite enough to give cause for reflection. He hopes he does not seem to be obstructive or over critical, but, from practical experience, he has reason to believe that a shield 42 ft. in diameter in this case presents unusual difficulties. With probably unusual knowledge as to the conditions of tunneling under the North River, conditions which thus far stand

* New York City.

alone, the speaker feels he is on well-known ground in dealing with tunnel problems under that waterway. Of the matters requiring notice he will mention only a few, and will refrain from detailed comment on the proposed shield and methods, beyond saying that he believes that Nature, as represented in atmosphere and geology, would not adjust itself to either in a way to conform with the practical requirements of tunneling. A shield and its tunnel lining constantly and harshly disagree; each must have a simple and tough constitution, free from pulmonary troubles suggested to the speaker by the proposed shield and conjunctive mining methods. Mr. Forgie.

On the New York side, at the Canal Street location, any tunnel, large or small, would be interfered with by a stretch of rock reef, and, the larger the tunnel, the greater would be the length of this interference, or tunneling in part rock overlaid with silt. The harbor dredging plane is 40 ft. below mean low water, and the minimum cover, in the speaker's opinion, should be about 20 ft.

With a tunnel 42 ft. in diameter, the length of this mixed ground to be tunneled through would be much greater than with a smaller or single-deck tunnel, and a greater differential air pressure would be required during the excavation of the rock reef. Any differential air pressure required would be 9 lb. greater than in a tunnel of the size of the Pennsylvania Railroad Tunnels, and those who are experienced know the difficulty of providing extra cover to meet such a condition and the trouble of keeping it in place.

The speaker is bearing in mind that this is solely a North River tunnel discussion, and that, while shoving a shield blindly through the silt, the air pressure used is generally the equivalent of a hydraulic head of from 5 to 10 ft. above the crown of the tunnel; but, also, when it is necessary to excavate in this selfsame silt ahead of the shield, the air pressure had to be about equal to the hydraulic head. The latter would be the air condition when excavating the rock reef, so that, if the air pressure required to dry out the ground over the rock surface near the invert of a 23-ft. tunnel was 6 lb. greater than is necessary at the crown, then, under similar conditions, this differential force would be 15 lb. in the large tunnel in question. This, added to the air pressure necessary at the crown, requires a somewhat impossible blanket cover.

This large tunnel would require a shield with a cross-sectional area of more than three times that of the Pennsylvania Tunnels and about seven times that of the Hudson and Manhattan Railroad Tunnels. In passing through silt, the latter were driven by displacing almost all the silt, and taking in little or none of it. This plan was tried on the Pennsylvania Railroad Tunnels in silt, but the grade could not be maintained, the tunnel rising unduly above it. Therefore, an average of about 35% of the tunnel displacement had to be

Mr. Forgie.

taken into the tunnel, at times none, at other times from 10 to 100%, but on the average, about 35% of the tunnel bulk throughout the entire length in silt. To the speaker these two cases indicate that the larger tunnel will require a greater proportion of silt to be taken in, and also creates questions in one's mind, concerning the propulsion as well as the steering of the shield in silt, which is governed greatly by the quantity of muck taken sausage-like through the shield.

Further, it is the speaker's opinion that a tunnel structure of reasonable cost in silt has to be flexible during the period of construction, the silt acting on it like a fluid, and the shield demanding that the lining follow it in what is never a straight path. A cast-iron tunnel circle, 23 ft. in diameter in the shield, becomes from 2 to 3 in. greater in vertical diameter, and correspondingly less in horizontal diameter, shortly after the shield leaves it; and, months later, it returns to its original shape. This distortion, in a tunnel of 23 ft. exterior diameter, will not be any less in one of 42 ft. exterior diameter, and in the speaker's opinion, it is not economically practicable to resist it structurally. With cast-iron lining, the plates are more or less free to slide on one another, and the character of the bolting permits further flexibility, though at times a segment is broken without either weakening or endangering the tunnel; with concrete blocks, this inevitable action has also to be borne in mind. Under the land the action is the reverse of that in the silt under the river, and was resisted by turnbuckles and posts.

It is essential that the tunnels be made as water-tight as possible, but it may not be difficult to decide whether this can be done most readily, economically, and permanently with a machined cast-iron structure or a concrete block structure.

The cost of a tunnel is greatly dependent on the speed of construction; practically the same over-all and labor costs have to be borne, whether the rate of construction is 6 or 16 ft. per day. Metal lining can be built very expeditiously by men having comparatively little experience or skill, and it is doubtful if any such performance could be approached with concrete blocks, in this case, because the speaker thinks the behavior of the shield in silt would not be very agreeable to that kind of lining, also in this case. The words "in this case" are repeated because the speaker wishes to see the use of concrete blocks, in shield-built tunnels, where safe and practicable.

Since the Compressed Air Labor Law and the war have greatly increased the prices of labor and materials, the speaker has not made a specific estimate for a tunnel under the North River. However, without apprehended difficulties, and with the assumption that the scheme is feasible, the known increase of difficulties, and the increased market rates of labor and materials, it is the speaker's opinion that the estimate of cost cited is insufficient to cover the plan under discussion.

Mr. Forgie.

The speaker has probably said enough to indicate that, in his opinion, this project of a 42-ft. tunnel requires more study; and that definite opinions cannot be given on this occasion nor in this way; besides, to him it has seemed that, on some matters, the premises are uncertain. The speaker has considered this discussion as informal, and has been pleased to join it in that spirit, though by no means irresponsibly.

Mr. Thomson has referred to the North River tunnels in silt as rising and falling with the tide, as does a floating body. That is the reverse of the case, as tidal rise (load) compresses the ground, and the tunnel, as a part thereof, goes with it, falling with the rise and rising with the fall a very little, as occurs in any ground structure subject to alternations of load. Briefly, in this connection, the tunnel does not move within the silt.

Mr. Parmley.

W. C. Parmley,* M. Am. Soc. C. E.—That the time has already arrived when there is pressing need for more ample traffic accommodations between New York City and the Jersey shore requires no argument. As a citizen and taxpayer of New Jersey for many years past, the speaker feels a personal interest in the matter of more adequate communication between the metropolitan center and the districts west of the Hudson River.

The 36-ft. tunnel plan advocated by Mr. O'Rourke certainly appeals to the imagination, and its great traffic capacity is a very compelling argument in its favor, provided the difficulties inherent in a tunnel of this size are not such as to offset its advantages.

The broad water barrier between the States of New York and New Jersey has already been set at naught by the completion of six tunnels, and there can remain no doubt as to the practicability of building more tunnels of approximately similar size and by similar methods; but as the proposed tunnel is nearly double the diameter of bore of any river tunnel yet built, and has nearly four times the area of heading, one wonders whether the difficulties of the kind encountered in the existing tunnels might not become embarrassing in a structure of this size.

The depth below the surface, necessary in order to give protecting cover, would be great, and also the pressures on the completed tunnel and the thrusts required of the jacks to push the shield ahead. In some places the ground is soft, approximating a hydrostatic condition, and at other places, in firmer material, there will be an entirely different distribution of the loads. These variations must be given full weight in the design of the tunnel section, but, before this can be done, it becomes essential to establish limiting relations between the vertical and horizontal components of the earth pressure. This relationship

* New York City.

Mr. Parmley.

also affects vitally the matter of pushing the shield and the ability of the newly erected tunnel lining to withstand the pressures without serious crushing, shearing, or splitting.

The published data in regard to driving the Pennsylvania Railroad Tunnels and the Hudson Tubes furnish important information on this subject. Messrs. Hewett and Brown, in their paper* on the Pennsylvania Railroad Tunnels, state that a number of the cast-iron segments lining the tunnel were broken by the unbalanced pressure of the ground. As this number was considerable (166 as reported), the conclusion is that they were being loaded well up toward their ultimate strength. If this assumption is correct, then, by finding the moment of resistance of the cast-iron lining and comparing it with the pressures and stresses due to earth pressure under different degrees of fluidity, one can deduce, at least, a rough idea as to the probable intensity of these pressures on a tunnel of the large diameter which it has been proposed to build.

As a check against this method of argument, the total thrust of the jacks in pushing the shield with all doors closed has been compared with the total combined weight of the earth and water over the tunnel, thus arriving at a ratio of static resistance of the earth to the total vertical weight. Then, by Rankine's formulas, the corresponding fluidity of the ground has been deduced. Without giving the details of these two calculations, it may be stated that they lead to substantially the same results. That at times the jacks were pushed to their limit is indicated by the statement of Messrs. Hewett and Brown, that the pistons had a tendency to bend, and that it would probably have been better if the diameter of the plungers had been 10 in. instead of $8\frac{1}{2}$ in. Furthermore, some of these same jacks were used for the shield of another tunnel, to which reference will presently be made, and it was found that, on account of bending, all had to be overhauled and re-aligned before they could be used.

In order to cover a wide range of softness of the earth, the pressures and moments of the 23-ft. Pennsylvania Railroad Tunnels were calculated for headings that would theoretically stand on natural slopes of 3:1, 4:1, 6:1, and 8:1. The last is the condition of hydrostatic pressure.

To shorten the story, Fig. 11 shows the equilibrium polygons for the four cases named, together with the approximate moments at the crown, the springing line, and the invert.

Now, taking into account the axial thrusts, and dividing each of these moments by the section modulus of the cast-iron lining segments at each of the points shown, the approximate stresses in the cast iron, for the different conditions of the earth, have been computed and are shown in Table 2.

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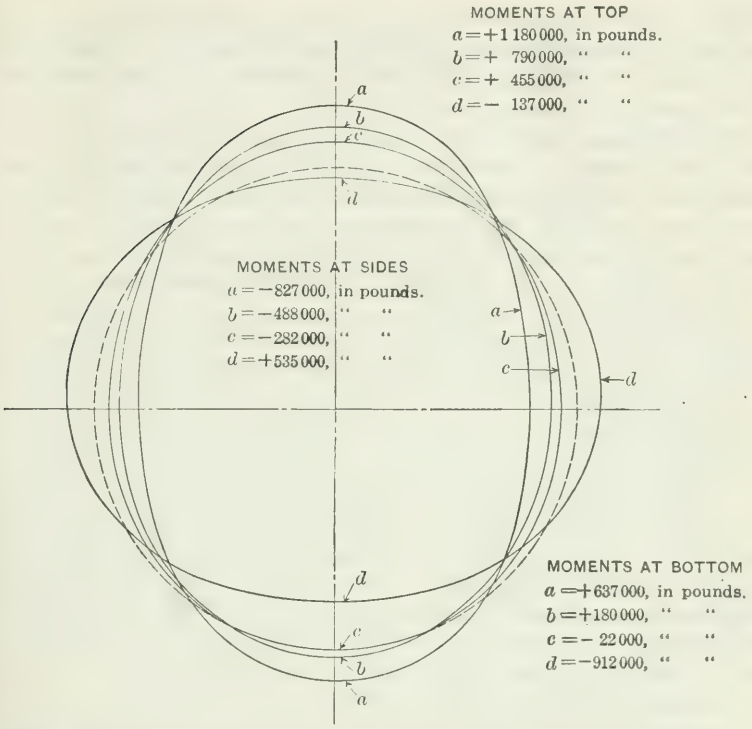
* *Transactions*, Am. Soc. C. E., Vol. LXVIII (1910), p. 152.


MOMENTS AT TOP
a = +1 180 000, in pounds.
b = + 790 000, " "
c = + 455 000, " "
d = − 137 000, " "
MOMENTS AT SIDES
a = −827 000, in pounds.
b = −488 000, " "
c = −282 000, " "
d = +535 000, " "
MOMENTS AT BOTTOM
a = +637 000, in pounds.
b = +180 000, " "
c = − 22 000, " "
d = −912 000, " "

APPROXIMATE EQUILIBRIUM POLYGONS FOR DIFFERENT KINDS OF MATERIAL SHOWN, AND FOR AXIS OF TUNNEL RING 39 FT. IN DIAMETER.

a for earth having natural slope, 3 horizontal to 1 vertical
b " " " " " 4 " " 1 "
c " " " " " 6 " " 1 "
d for Hydrostatic (fluid), " ∞ " " 1 "

FIG. 11.

TABLE 2.—APPROXIMATE TRANSVERSE STRESSES IN CAST-IRON LINING, IN POUNDS PER SQUARE INCH.

Material.	Top.	Sides.	Bottom.
3 : 1	+ 25 300	− 16 100	+ 13 000
4 : 1	+ 16 300	− 9 600	+ 38 000
6 : 1	+ 8 600	− 4 000	+ No tension.
Hydrostatic pressure	− 900	+ 9 700	− 17 900

Mr. Parmley.

The algebraic signs in Table 2 indicate on which side of the axis the line of pressure comes, the plus sign indicating tension on the inner side of the ring and the negative sign tension on the outer side of the segment.

The different values for the maximum stresses correspond well with the known values for cast iron, but it is interesting to note that those found by A. N. Talbot, Past-President, Am. Soc. C. E., for cast-iron pipes tested by him, which would provide conditions more closely applicable to this case, as they were obtained from curved instead of from straight specimens, ranged from 21 000 to 33 000 lb. per sq. in.

From an inspection of the foregoing results, taken in connection with the actual breakages, it seems plain that the ground has natural slopes of approximately 3 : 1 or 4 : 1, as most of the breaks were in the top of the tunnel, with a few in the invert, indicated by Table 2 as the points producing the most intense stress, and these stresses in turn correspond with the moduli to be expected in the cast-iron segments.

Interest in the hydrostatic condition lies mainly in showing the direction in which the variation of stress travels as the ground becomes more and more fluid. It would also appear that both the vertical and horizontal pressures are functions of the depth, and that the limits of horizontal to vertical intensity probably lie between about 50% for the case of 3 : 1 material, and about 60%, for the case of 4 : 1 ground, results which also agree with values obtained by taking account of the pressures required of the jacks, as already stated.

Although it is probable that the air pressure, due to its drying-up effects on the earth, causes the latter to offer a greater horizontal resistance to the shield movement, and correspondingly less horizontal pressure on the tunnel ring than it naturally does, the error, if any, in both cases, is on the side of safety in these calculations.

It would appear from the foregoing that, including the effect of air pressure it would be necessary to provide ram power equal to the static resistance of earth having a natural slope of 2 horizontal to 1 vertical. This requirement would give to the Pennsylvania Railroad Tunnel shield 3 150 tons, where actually 3 300 tons were provided. In similar manner, the shield of the Hudson Tubes would have required about 1 600 tons, where 2 000 tons were actually provided. Reduced to a square-foot heading basis, the effective push of the jacks in the Pennsylvania Railroad Tunnel amounted to about 8 tons and in the Hudson Tubes to about 9 tons.

The conclusion, therefore, is that in any large-size tunnel, such as is now proposed, because of its greater depth and the increased possibility of encountering denser material, not less than 10 tons per sq. ft. of heading should be provided.

If this conclusion is justified, one is led naturally to ask the following questions, and on their affirmative answer depends the possible success or failure of a concrete lining in such a tunnel as now proposed. Mr. Parmley.

1st.—In a tunnel of 36 ft. interior diameter, what thrusts and bending moments will the ring be subjected to, and will a 36-in. thickness of ring be sufficient?

2d.—Will the 36-in. ring be able to carry the thrusts of the powerful jacks that it will be necessary to use?

3d.—Will the concrete lining courses be able to resist the shearing and bending produced by the jacks?

In answer to the first question, it may be stated at once, without going into detail, that a 36-ft. tunnel, 3 ft. thick, with 25 ft. of earth and 45 ft. of water above its top, and with earth of a character which gives a horizontal pressure equal to 50%, or less, of the vertical, the crushing and bending stresses on an unsupported ring would be destructive. At the crown the thrust would be about 85 000 lb. per lin. ft. of tunnel, combined with a moment of more than 3 400 000 in-lb. If acting on a plain concrete section, even though no joints occurred at the crown, it would mean a crushing force on the outer edge of the ring of 2 500 lb. per sq. in., and in tension on the inner edge of more than 1 100 lb. per sq. in. The corresponding stresses at the springing line and the invert would also be destructive. More extended calculations for the thrusts and moments at the critical points, for the variations in the character of the ground noted, show that a ring, 48 in. thick and heavily reinforced, would be necessary to meet the conditions safely.

With a ring 36 in. thick it becomes absolutely essential, therefore, that it be supported and fixed at several points by interior walls and floors. Only in this way can the line of resistance be kept near the central line and safe pressures on the concrete be secured. The same statement also may be made regarding a cast-iron segment ring of reasonable thickness.

It should be noted here that the upper horizontal floor, occurring at about the upper quarter points, should be made to act primarily as a tension member, and that additional strength should be given to the inside of the tunnel ring at the crown by suitable reinforcement or bracing. The invert, also, would need similar attention. All points of support for walls and floors must be designed to become effective immediately after the shield is pushed off the block rings, otherwise, destructive effects might be experienced. If, however, these provisions are properly taken, the ring, in so far as these factors are concerned, may be made safe, and the first question may be considered as answered satisfactorily.

Mr. Parmley.

As to the second question, regarding the pressure of the rams on the concrete itself: If a pressure of 10 tons per sq. ft. is used for the 42-ft. heading under consideration, the total pressure becomes 13 800 tons, and this, if distributed uniformly around the center line of the concrete ring, requires 111 tons per lin. ft. of the circumference. If the ring is made up of sixteen blocks, as shown in the published plans for the tunnel, there would be thirty-two jacks of 430 tons each. At a pressure of 5 000 lb. per sq. in., the plungers would be 15 in. in diameter—rather formidable looking affairs, to say the least. They would be 3 ft. 10 in. from center to center around the center line of the concrete ring. Now, if it were possible to distribute this pressure evenly over the entire area of the face of the concrete ring, the pressure on the concrete would still be 520 lb. per sq. in. Such even distribution, of course, is impossible.

If a system of lug contacts between the blocks is used, in order to prevent beam and shear effects, it would be necessary to limit the number of contacts to two per block. These contacts, therefore, would of necessity be of considerable size, or the excessive concentration of pressure would result in splitting and bursting the blocks. To increase considerably the areas of these contacts would result in the same difficulty in distribution as in the case of plain block-to-block contacts; otherwise, bursting concentrations would result. That a large amount of distribution is necessary is evident from the pressure given for the case of complete ideal distribution.

The only adequate way out of the difficulty seems to be to use a series of aligned cores extending through the body of the blocks from front to back, and against which each jack may re-act directly. These cores can be made of high resisting material, slightly longer than the width of the blocks, so as to operate loosely through the blocks, or, they may be manufactured in the blocks and retained therein by frictional means. They may be made of a rich mixture of cement inside a heavy steel hooping, or completely encased in iron pipe. In this way a compressive resistance of 5 000 lb. per sq. in., equal to that of the jacks, or even more, could easily be secured. They would have the advantage that any crushing effect would not affect the block proper, and need not throw any strains on the blocks, due to irregularities in their position in the ring.

They can also be made with a small cap at the end, of weaker material, which will crush and thus automatically give the needed distribution of pressure over the end of the core. They may also be made to serve most effectively to pin and lock the blocks of one ring to those of the adjacent rings, as the contact of core to core can readily be made to occur either between the rings or within the body of the block itself.

Mr. Parmley.

A further and very great advantage of the core method lies in the possibility of applying four instead of only two jacks to each block. With fixed bearing surfaces, it is not possible to use more than two contacts to the block, because that would imply absolutely perfect bearing of ring to ring, but, if the cores are used to transmit the reactions independently of the ring blocks, the number may be increased to any extent desired.

With four jacks per block, the number is increased to sixty-four, and the diameter of each is reduced to 10½ in., with a maximum of 215 tons per jack. These jacks would now be more in line with the sizes already used, and the fact that the cores may be permitted to slip in the blocks and adjust themselves would permit equalization of the pressures and result in a completely interlocked lining, sound and suitable to withstand its proper burden. The second question, therefore, may be considered as answered in the affirmative, if these precautions in the design of the ring are taken.

Taking up the third question, in regard to shearing stresses, it is obvious that the shear which otherwise would be sustained by the blocks is now transferred wholly to the cores. That this precaution is absolutely essential may be shown by a simple calculation.

Suppose there is a ring of blocks, 36 in. thick and 18 in. wide, or a sectional area of 648 sq. in. per ring of block tunnel. Consider a block lining in which the longitudinal joints are broken. In the absence of perfect bearing of block against block along the entire length of the circumference, one of the jacks bearing on the block will act against an overhanging end of one-half the length of the block, thus setting up both bending and shearing stress in it. A 430-ton pressure would thus give a mean shear over the cross-section of the block of 1 380 lb. per sq. in. It is thus evident that the block would fail, either in bending or shear, almost before the jack had been adjusted to its serious work.

On the other hand, by using suitable cores, the shearing stresses in the block can be practically avoided, and, in so doing, it is also relieved of bending stress.

A statement was made in a New York paper of March 16th, 1919, to the effect that the use of concrete blocks would be experimental, as this method had never been used before. This statement is in error. Sewer tunnels of concrete block construction, designed by the speaker, and costing hundreds of thousands of dollars, have been built with entire success, and have been substantially water-tight. The new water-works tunnel at Cleveland, Ohio, costing upwards of $1 000 000, with which the speaker was connected in a consulting and constructing capacity, was built successfully with reinforced concrete blocks. It is 10 ft. in inside diameter and is built of rings of reinforced blocks 11 in. thick. The length of the tunnel is about 15 000 ft., and its

Mr. Parmley.

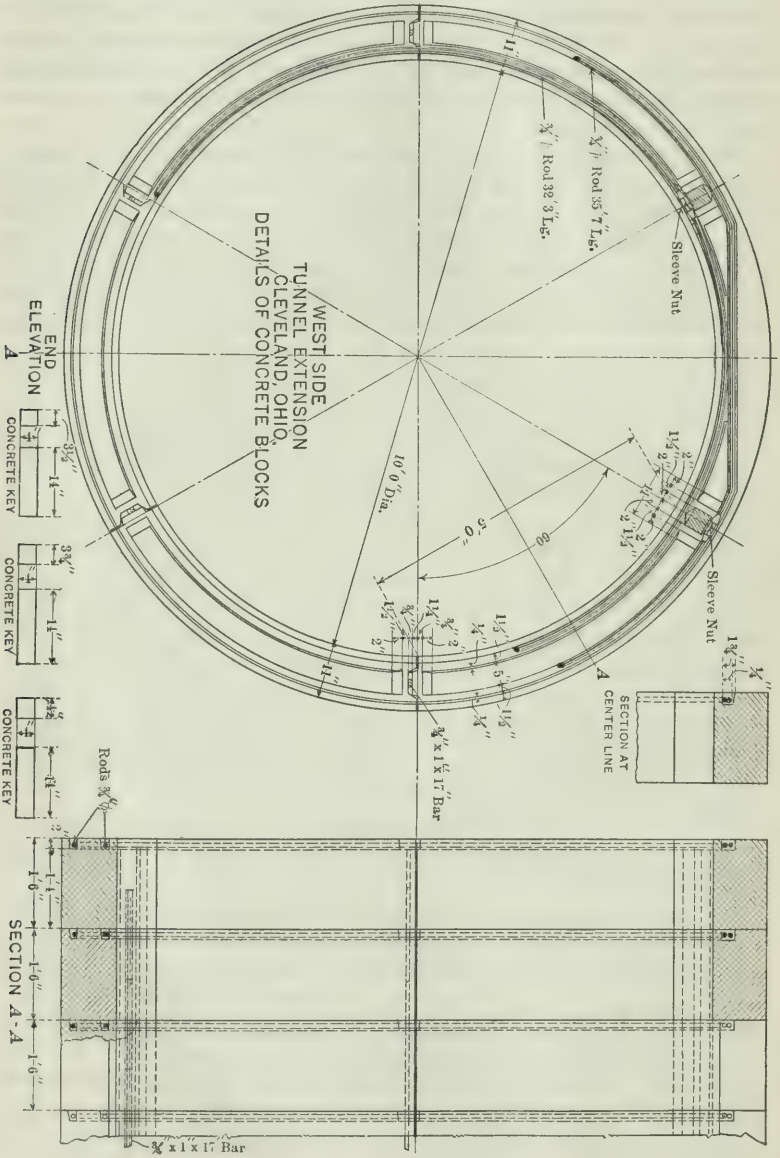
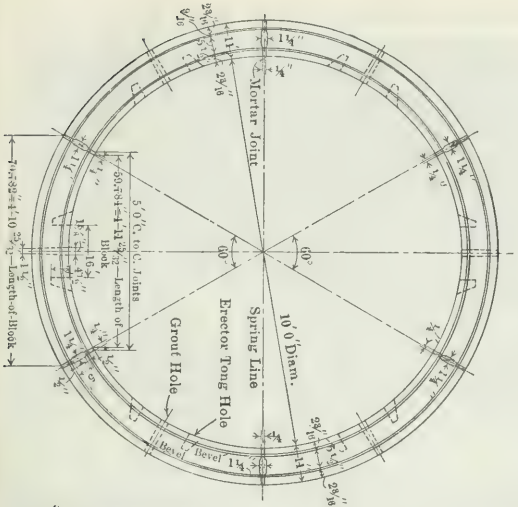
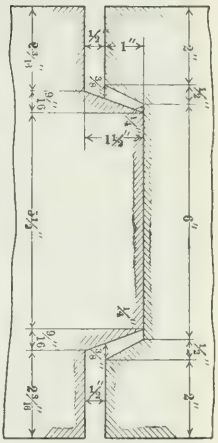
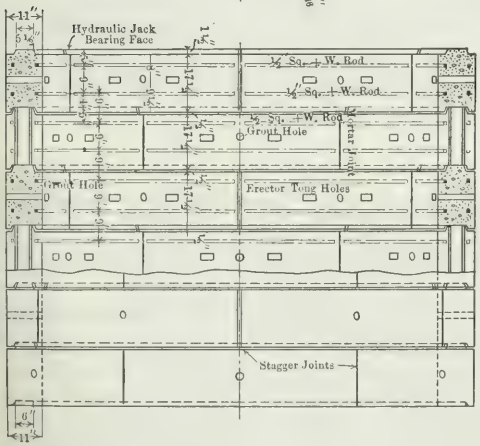


FIG. 12.

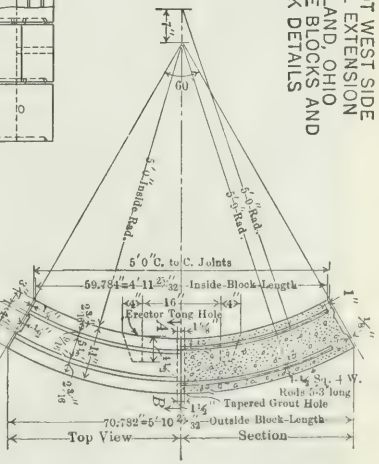
Mr. Parmley.







TEN FOOT WEST SIDE TUNNEL EXTENSION CLEVELAND, OHIO CONCRETE BLOCKS AND BLOCK DETAILS



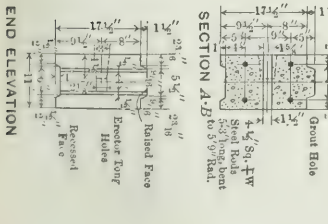


FIG. 13.

Mr. Parmley. center is about 95 ft. below the surface of Lake Erie for the entire distance. Therefore, it carries a burden of from 40 to 50 ft. of soft clay and sand overhead and about 50 ft. of water, for its entire length.

For most of the work, an air pressure of from 15 to 25 lb. was used, and the shield, which was about 12 ft. 2 in. in diameter, was driven forward by twelve 8½-in. jacks, which, at 5 000 lb. pressure per sq. in., gave a thrust of about 137 tons each, or a total of 1 650 tons. The speaker believes that they were never driven to their limit. In order to start the shield, about 3 000 lb. pressure was used, but, as soon as it moved, the pressure was reduced, and only about half as much was necessary to maintain the forward movement.

At a pressure of 3 000 lb. per sq. in., and considering full and ideal distribution of pressure on the block ring, a thrust of about 430 lb. per sq. in. against the concrete blocks was developed. Of course, it was impossible to get this ideal distribution. Part of the tunnel was built with continuous longitudinal joints, and part with the longitudinal joints broken.

At the pressures mentioned, some spalling of the blocks resulted, and, in the portion laid with longitudinal joints broken, there was still more spalling, and a number of blocks were sheared. It is evident that, if the jacks had been worked to their limit, considerably more damage to the blocks would have resulted, even with the best distribution of the pressures possible by means of wood blocking.

It was the behavior of concrete blocks under jack reactions, as thus actually observed, and the damages resulting, which convinced the speaker that, in order to insure the success of concrete block lining for this undertaking, it will be essential to take the precautions which have just been discussed.

Whether the tunnel is to be a single barrel of large dimensions, divided up for the different roadways, or twin tunnels of smaller diameter, it will be necessary to take such precautions, but, if they are taken, there seems to be no reason for not using a concrete block lining with entire success and safety, without involving the large expense of cast-iron or cast-steel segments. The tunnel can be made water-tight by the grouting process. Its construction involves the entire elimination of one major process—the cost and erection of the cast-iron segments—and the concrete in the blocks will be of a far higher quality and uniformity than can be obtained in practice where the concrete is deposited in mass within the tunnel.

Mr. Thomson. T. Kennard Thomson,* M. Am. Soc. C. E.—Not one, but several, tunnels should be constructed between New York and New Jersey.

The ventilation can be easily taken care of; but separate compartments should be provided for foot passengers and horse-drawn vehicles, and for fast and slow motors.

* New York City.

FIG. 14.—CONCRETE BLOCKS FOR LINING CLEVELAND WATER SUPPLY TUNNEL.

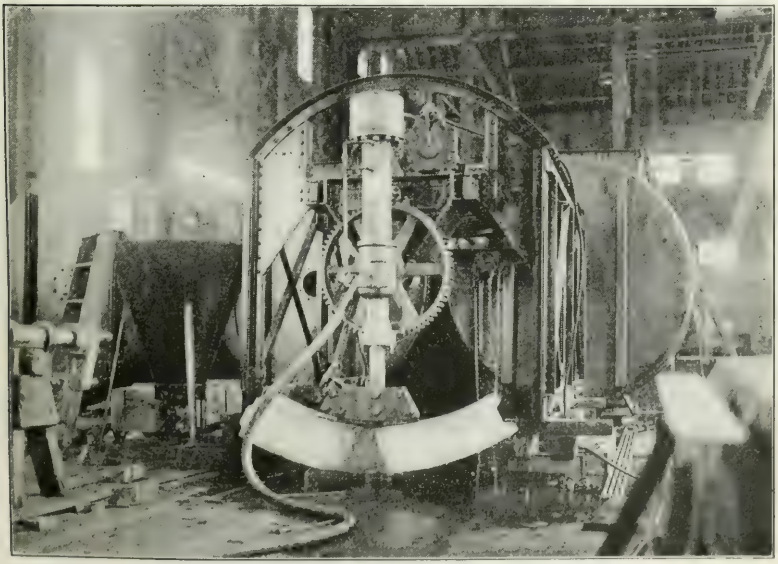


FIG. 15.—MACHINE FOR SETTING CONCRETE BLOCKS FOR CLEVELAND WATER SUPPLY TUNNEL.

Mr. Thomson

The first requisite for a tunnel should be safety; the second, permanence; and the third, economy.

A shield-driven tunnel under the Hudson River does not, and will not, comply with any one of these three requirements, as compared with what can be done by the trench method. The reason for this is that the Hudson River silt is about 200 ft. deep, and any tunnel in it must perforce be a floating tunnel. For, if its weight is made just right without any live load, it will be too heavy with trains or vehicles in it. In short, it is a physical impossibility to design a tunnel which would always be in equilibrium. As a matter of fact, the Pennsylvania Tunnel, probably the best lined tunnel in the world, rises and falls inversely as the tide.

James Forgie, M. Am. Soc. C. E., in a remarkably complete monograph* on this tunnel, has given a very good description of the value of this silt for bearing purposes. He explains that the shield, 23 ft. in diameter, could have been driven across without excavating any material whatever, except for the difficulty of keeping the tunnel in its proper location, so, as a matter of fact, from 30 to 60% of the material was excavated, and the rest was squeezed out of the way. Who would ever think of building a house on material that could be squeezed so easily? He states, further, that the tunnel itself was subject to distortion when a second tunnel was being driven alongside.

Now, a tunnel that rises and falls inversely as the tides, as perforce any tunnel must which is shield-driven through this material, would be likely sooner or later to get a water-jacket around it, and, if this ever happened, a catastrophe might result, as the tunnel lining, as usually built, would not stand much tensile strain, especially in a longitudinal direction.

The proposed concrete blocks would not be nearly as strong as the lining of the Pennsylvania Tunnels, which is composed of heavy cast iron with 2 ft. of concrete.

Surely it would be better to construct a tunnel which would always be rigid and in a state of equilibrium, and would remain permanently where put. This can be done easily by the open-trench method, that is, by dredging a channel and driving an ample number of foundation piles, so that the tunnel could never sink any lower, and then make the tunnel so heavy that it could never rise, and so wide that it could never bend. It could be built on the surface, like a boat, with bottom, sides, and roof of reinforced concrete, say, 6 ft. thick. It could be in lengths of 1 000 ft. or more, and when ready could be towed into position and easily sunk on the piles. An air chamber could be provided below it, in order to permit any additional excavation, if desired, after the section had been sunk.

* *Engineering News*, February 28th, 1917, p. 228.

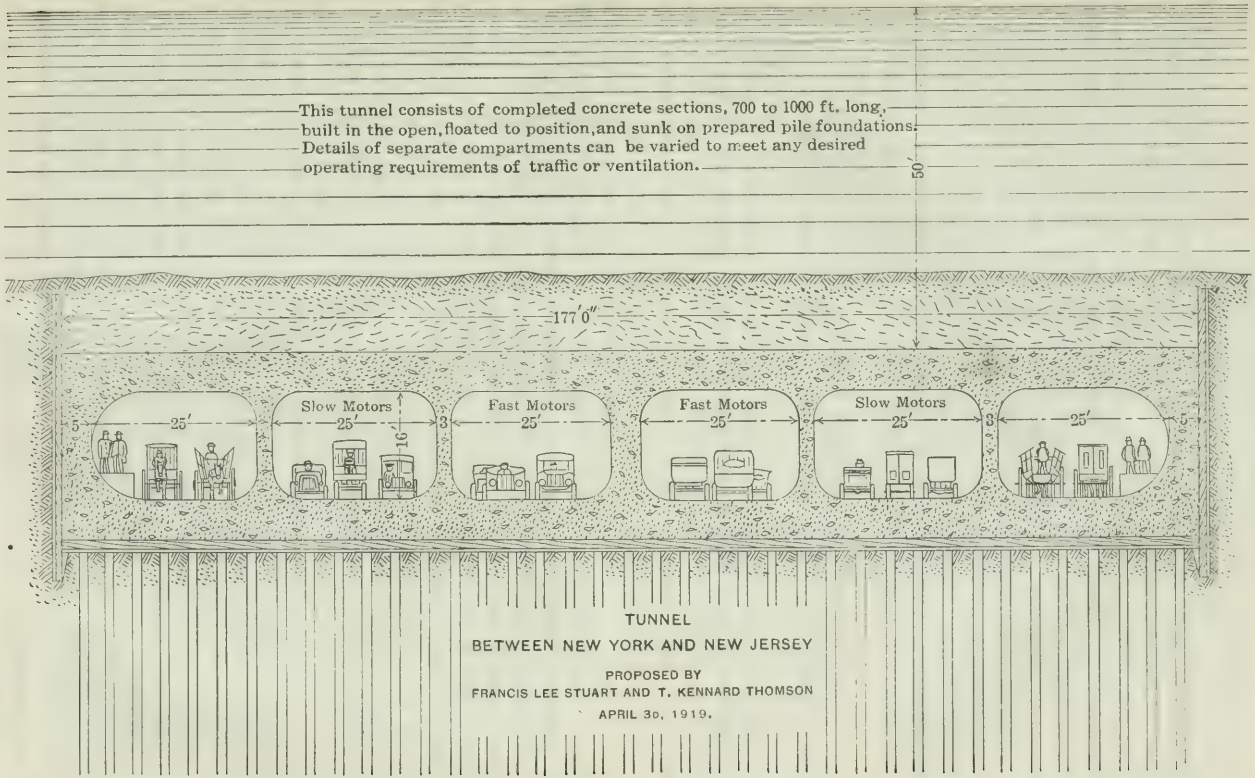

This tunnel consists of completed concrete sections, 700 to 1000 ft. long,
built in the open, floated to position, and sunk on prepared pile foundations.
Details of separate compartments can be varied to meet any desired
operating requirements of traffic or ventilation.
50'
177' 0"
Slow Motors
Fast Motors
Fast Motors
Slow Motors
25'
16'
TUNNEL
BETWEEN NEW YORK AND NEW JERSEY
PROPOSED BY
FRANCIS LEE STUART AND T. KENNARD THOMSON
APRIL 30, 1919.

FIG. 16.

Mr. Thomson.

There are many ways of making the joints between the sections after they are in position, but the speaker's method would be to sink pneumatic caissons over the joints. Mr. Thomson.

The proposed 42-ft. tunnel has two compartments, one for each direction. That means that horses and fast and slow motors going in one direction would all travel in one compartment, thus preventing real speed. The trench method would permit of six compartments, each larger than those proposed for the shield-driven tunnel. In this way horses, slow motors, and fast motors would be separated from each other by walls of concrete 3 ft. thick.

Obviously, a trench tunnel can be placed at a higher level than a shield-driven tunnel, especially in such fluid material, thus shortening the approaches by 1 000 ft. or more, which is a great advantage. Such a trench tunnel, Fig. 16, is proposed by Francis Lee Stuart, M. Am. Soc. C. E., and the speaker, and would have three times the capacity of the shield-driven tunnel, without costing any more to construct, and be safe beyond peradventure of a doubt, as no shield-driven tunnel ever can be without going to unheard of expense.

F. Lavis, M. Am. Soc. C. E.*—This discussion has been very interesting. There can be little doubt that it is desirable to establish communications between New York City and New Jersey by means of a vehicular tunnel, or even several of them. The question of design, however, needs serious consideration, and the speaker does not believe that this is the place to decide the merits of the various proposals. The salient points which affect a decision as to the proper design have already been pointed out, and the speaker does not intend to go into that matter further. Mr. Lavis.

Mr. O'Rourke's section has many features to commend it, if it can be built, but it certainly is a most serious matter to commit the future of an enterprise like this to a type of construction which presents so many serious difficulties of execution, and which has not been tried out in some way.

It is a long time since the speaker has had anything to do with actual tunnel construction, and he is inclined to look at all these matters from the standpoint of people who are asked to provide the money for various enterprises. He is inclined to think, therefore, that if the construction of a vehicular tunnel to New Jersey were to be carried out by private enterprise, there would be, to say the least, a great deal of hesitation in making a commitment, involving some $12 000 000 at the lowest estimates, with so many uncertainties involved.

Of course it is realized that, if no one had ever ventured into the unknown, there would never have been any shield-driven tunnels, or much of anything else, and the speaker does not wish for a moment to say that Mr. O'Rourke's scheme cannot be carried out. It is realized,

* New York City.

Mr. Lavis. also, that it is not altogether the same to advocate the expenditure, in some cases, of public money to aid in the progress of the Arts and Sciences; so that the use of public funds for the purpose cannot be put on exactly the same basis as the use of private capital.

Just at the present moment, however, in view of the tremendous demands for capital all over the world, demands which will exceed the supply for so many years in the future that we cannot see the end, it behooves every one to be rather more conservative than usual. At any rate, if the section proposed by Mr. O'Rourke is so much more desirable than others, if the cost of its construction—in case of the success of his scheme—is thought to be enough less than others to warrant it, the logical thing to do would seem to be to make the experiment, at least on a large enough scale to determine its feasibility, before commitment to the whole enterprise.

Now, aside from the merits of one scheme or the other, there is the proposal of Mr. Brown that this meeting should decide the matters of controversy. The speaker does not think it is at all the function of this Society as such, or of the meeting, to advise any one on a proposed engineering project relating to civil affairs. During the war all have given their time and services to the Government, in so far as they have been required, but, in a matter of this kind, which is one of ordinary civil and commercial life, the speaker would suggest that it is a proper subject for a properly appointed commission of qualified engineers, who should be paid for their services and for their knowledge. The camouflage of the word "reconstruction" ought not to cover the giving of engineering advice gratuitously to the Government—Municipal, State, or National.

Mr. Tomkins. Calvin Tomkins,* Assoc. Am. Soc. C. E.—Transportation between New York and New Jersey is the crux of port organization in New York City. It is not a local problem, but extends from New York City to Albany, for, with the exception of one bridge of insufficient capacity (that at Poughkeepsie), there is no all-rail crossing. The consequence is that, to the already congested lighterage transfer at the Port of New York, there is added the large transfer of continental business between New England, Canada, and the remainder of the United States. In some way improved all-rail communications between these parts of North America must be secured, and thus avoid the convergence of the congestion at the Port of New York. For passenger service this has been accomplished with the tunnels of the Pennsylvania Railroad, and now the same thing must be done for freight.

The vehicular tunnel project is commendable as a first step toward solving the freight problem, but it still involves the necessity of breaking bulk in New Jersey and the transfer of freight by truck or lighter to New York City. That, indeed, is better than the present

* New York City.

Mr. Tomkins.

method of effecting transfers between New York and New Jersey, but it necessitates the continuance of the railroad terminals in New Jersey, and, on that account, is not economical. In addition, a vehicular tunnel will only provide for service to Manhattan; it will not reach to Long Island, except across the bridges.

For motor truck traffic the tunnel is all right, but the all-rail connection is needed, not only with Manhattan, but across that Borough to New England and to connect with the Canadian Railroads. In other words, if New York City is to obtain the greatest port advantages possible, there must be done for freight what the Pennsylvania Railroad has done for passenger transportation. The vehicular tunnel project is welcomed as a step in the right direction, but only as a beginning.

When the railroad tunnels for freight have been completed, the conditions in New York City will be comparable to those which obtain at every other port in the world. The port is unique in being divided into two parts by the Hudson River barrier, which necessitates an immense transfer by floating equipment. This caused the coal famine in 1917-18, and is responsible for the strike in 1919, because the nice adjustment of the intricate floating machinery can easily be made unworkable by labor disturbances.

This great floating transfer must be abolished. Every other port in the world is accessible from all sides by railroads which lead to those ports, and such railroads can be made to connect with each other and with the docks, warehouses, and factories by a loop system of terminal railroads. Everywhere, except only at the Port of New York, this kind of railroad terminal distribution is in operation. Since the war placed the railroads under one administration, correct railroad integration has been established at all United States ports except only New York. At that port the Government is now largely in control of the transfers by water, and much of the former friction and expense incident to private separatist control of equipment has been done away with. When the Long Island and New Jersey systems are connected, and, incidentally, Manhattan is connected with both, freight can be transferred cheaply and advantageously from one side of the harbor to the other.

The steamships, factories, population, and wealth, are, for the most part, on the east side of the port, and the greater part of the railroad terminal system of the country is on the west side. The two parts must be joined, and thus make transportation a continuous process.

Let this tunnel project, then, be regarded as the first important step toward a complete union of the two sides of the port, which will make New York like other great seaports and remove from it the unenviable distinction of being the worst organized of the large ports of the world.

Mr. Schaeffer.

AMOS SCHAEFFER,* M. AM. SOC. C. E. (by letter).—It is not clear why a tunnel 42 ft. in diameter should be built, even if it is possible to do so. All the economies which are claimed for a tunnel of this size could be applied to one of smaller diameter. Practically all the quantities will be greater for one large tunnel than for two small ones. The shield will weigh more and cost more to build, the quantity of material to be excavated will be greater, and the quantity of concrete for the block lining will be greater. As the number of points of attack for two smaller tunnels would be greater, and the material would be handled through double the number of shafts and by double plants, two tunnels could be driven much more rapidly than one of the larger size.

Therefore, estimates of cost, as between one large tunnel and two small ones, should be based on the use of the same materials. If it is practicable and economical to use concrete block lining in the large tunnel, it is likewise practicable and economical to use it in the smaller tunnels. On this basis, two smaller tunnels, without doubt, could be shown to cost less than the larger one.

It is not intended to enter into a discussion on the merits of concrete block lining, but simply to point out that if its use is practicable in one case it is in the other, and, therefore, that it is unfair to compare the cost of a large tunnel lined with concrete blocks with that of two smaller ones lined with cast iron.

The capacity of the proposed tunnel is greater than necessary for the present needs of traffic and those of the immediate future. It provides for two levels, with three lines of travel moving in the same direction on each level. A capacity of 100 000 vehicles per day is claimed. From the best information available, this estimate is based apparently on the following rates of speed and distances apart of vehicles:

3	miles	per	hour,	50	ft.	apart.
9	"	"	"	60	"	"
15	"	"	"	80	"	"

These assumptions require a 24-hour day to provide a capacity of 100 000 vehicles.

It is believed that at least two of these assumptions need revision, in order to make a determination of the maximum capacity. A 12-hour day is the longest that should be used as the time basis. It is well known that traffic will not use a highway to its maximum capacity for 24 hours continuously.

In view of the distance within which motor vehicles, moving at various rates of speed, can be stopped by the application of brakes, the spacing of vehicles can be reduced.

A horse-drawn vehicle, moving at the rate of 3 miles an hour, can be stopped within a distance of a very few feet. Motor vehicles moving

* New York City.

at the rate of 9 miles an hour can be stopped within a distance of 9 ft., and those moving at the rate of 15 miles an hour can be stopped within a distance of 21 ft. With a margin of safety of not less than 100%, these spacings can safely be reduced to 30, 40, and 60 ft., respectively. Mr. Schaeffer.

On this basis, with the same rates of speed, and a 12-hour day, the maximum capacity of a tunnel with six lines of traffic would be 37 000 vehicles in one direction, or 74 000 in both directions.

It has been assumed that, at the completion of the tunnel, say, in 1922, 4 000 vehicles per day would use it in each direction, or 8 000 in both directions. This would be only about one-ninth of its capacity. The Commission assumes an annual increase in traffic of 5 per cent. This rate of increase was used undoubtedly more to show the small amount of traffic which would produce sufficient revenue to pay the interest, amortization, and maintenance charges than to indicate the traffic which could be expected.

On the assumption that traffic would increase at the rate of 8% per annum, there would be 37 160 vehicles using it per day in 1942, or 20 years after its completion, which is still only one-half its maximum.

It would seem, therefore, that a tunnel with a diameter of 42 ft. is probably considerably larger than is needed at first.

Two tunnels with a capacity for two lines of traffic, traveling at the rate of 3 and 9 miles per hour, and with 30 and 40 ft., respectively, between vehicles, would have a maximum capacity for a 12-hour day of about 41 000 vehicles in both directions, which is five times the capacity required at their completion, and would be ample for the traffic in 1942, 20 years after their completion.

It is admitted, of course, that six lines of traffic in three tunnels will not give as large a capacity as six lines in one tunnel on two levels, but it is believed that the smaller cost of two tunnels and the postponement of the construction of additional tunnels until they are needed, will largely compensate for this.

There is still another, and perhaps the most important reason, for constructing two smaller tunnels instead of a large one, namely, traffic conditions in Manhattan.

With the experience which New York City has with traffic, the concentration of six lines of vehicles moving in opposite directions, within a small area, cannot be considered good planning, unless there are overwhelming reasons for it, such, for example, as the provision of additional roadways on the East River bridges, where the cost of construction to support the dead load is so much in excess of that to support the live load that the cost of providing for additional lines of traffic is relatively small.

In tunnel construction by the shield method, the cost of providing for additional lines of traffic will be more nearly in the ratio of the additional facilities.

Mr. Schaeffer.

With the entrances and exits as close together as they are shown on the plan, there would undoubtedly be confusion of traffic on account of its movement in opposite directions and congestion on account of its volume within a small area.

This objection can be overcome by locating the entrances of the smaller tunnels a block or two apart. If future tunnels are needed, it would be preferable to locate their entrances some distance from those already constructed on both shores. It is not conceivable that all traffic would have Canal Street, Manhattan, or 12th Street, Jersey City, as its objective points. Traffic, therefore, would be served very much better if additional tunnels were built at some point farther up the river, say, at about 14th Street.

In the latter part of 1915 the Police Department took traffic observations at street intersections at more than 200 points in New York City. This count showed a traffic ranging from 29 200 vehicles in 10 hours at Columbus Circle, to 7 000 vehicles in the same time at Madison Avenue and 59th Street.

In 1908, the north- and south-bound traffic on Fifth Avenue at 42d Street was 8 945 during a period of 10 hours, and, in 1916, it was 16 860 for the same time, showing an increase of 90% in 8 years. As everybody is familiar with the congestion of traffic at this point, these figures will serve as a measure of traffic at other points.

The traffic which may be expected to use the tunnel in 1942 has been estimated at 37 160 vehicles, which is exclusive of the normal traffic which will use the streets at and about the entrance to the tunnel. This is more than double that which existed on Fifth Avenue at 42d Street in 1916 and is only 50% of the capacity of the tunnel. In other words, the construction of a tunnel of the capacity proposed would create a congestion of traffic greater than that for the removal of which the city is at present expending approximately more than $625 000 in the construction of the ramp at 34th Street and the Viaduct between 40th and 42d Streets on Park Avenue. These improvements are made primarily to provide a thoroughfare on Park Avenue for the purpose of relieving congestion of traffic on Fifth Avenue. Before the full benefits from this improvement can be derived, it will be necessary to acquire the right to use the roadway around the easterly side of the Grand Central Station, which was formerly Depew Place. It is estimated that the cost of acquiring this right and making the necessary changes will involve an additional sum of $625 000, or a total of $1 250 000.

It seems inconsistent, therefore, to create a condition, at large expense, in one part of the city for the removal of which large sums are spent in another. It would seem that there are abundant reasons, aside from the structural uncertainties in a large tunnel, which indicate the construction of smaller tunnels than the one proposed.

E. E. R. Tratman,* Assoc. M. Am. Soc. C. E. (by letter).—With reference to the accommodation to be provided in the proposed Hudson River Tunnel for vehicular traffic, it appears to the writer that more attention should be given to that for pedestrians. Although no large amount of pedestrian traffic may be expected in a tunnel of such length, there are special conditions and emergencies to be considered. For instance, there is the possibility of a vehicle breakdown or blockade, causing passengers to leave cars or omnibuses and start to walk out of the tunnel. If a blockade should be attended by an accident, such as a collision or the burning of an automobile, there is the further possibility of a panic or a rush for escape. None of the plans appears to provide adequately for such contingencies. Mr. Tratman.

In the proposed 31-ft. tunnel section, with two sidewalks on opposite sides of the upper deck roadway, the available width of these walks is limited materially by the overhead curve of the tunnel wall, as the sidewalk level is slightly above the center line of the tunnel. This would cause people instinctively or necessarily to keep near the curb. As a matter of fact, it does not seem that two people could walk side by side or pass each other without inconvenience.

In the Blackwall Tunnel under the Thames, at London, the sidewalk level is about 4 ft. below the center line, so that the tunnel wall curves outward from the walks and there is ample overhead clearance. This is also true of the Rotherhithe Tunnel, with its greater diameter and greater width of sidewalks. It is not easy to understand the reason for the inner curb or footguard on the sidewalks, which forms a part of the lining in these two tunnels. It reduces the effective width by a few inches, although every available inch is desirable. In fact, the capacity and convenience of the walk might be increased by flattening the curve of the tunnel lining for the height of a few inches above the walk, thus increasing the actual width. This would not affect the strength of the tunnel.

In the proposed 42-ft. section for the Hudson River Tunnel, the single depressed footwalk appears to be inconvenient and inadequate, with the further objection that dust and dirt from the roadway are likely to be scattered over the pedestrians. The plan for a multiple-tube concrete tunnel also shows scant accommodation for pedestrians, although the designers note that details of the compartments can be varied to meet requirements.

Ventilation of the different levels and compartments will be a special problem in such a long tunnel having traffic composed largely of motor vehicles, and in which the vehicles will occupy so high a proportion of the cross-sectional area of the compartments. For instance, in one design the high-speed compartment has a headway of 9 ft., while the height of the highest vehicles is said to be 7 ft. 10 in.

* Wheaton, Ill.

Mr. Tratman. Dilution by forced draft would seem to be necessary, probably in combination with an exhaust or induced draft.

About a year ago, it was proposed to extend a street across a Chicago park by means of a tunnel, in order to avoid cutting the park by a surface thoroughfare. One question was in regard to ventilation conditions with a heavy traffic of motor vehicles. The writer made some inquiry as to conditions in the two Thames Tunnels at London, but the motor traffic was so slight, comparatively, that pollution of the atmosphere from that cause was a very small factor. Apparently, those tunnels have provision for artificial ventilation, but this is not in operation. Further, the cross-sectional area is much larger in proportion to that of the vehicles, than is the case in the proposed designs for the Hudson River Tunnel.

The double-deck tunnel idea is of particular interest to the writer for the reason that about 1890 or 1891 he undertook to suggest such an idea to the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., then Consulting Engineer for the proposed Blackwall Tunnel under the Thames. An oval section was proposed, with a diaphragm or upper deck for the roadway and a narrower bottom deck for pedestrians. Naturally, the reply was to the effect that for a tunnel in loose material nothing but a circular section could be considered. It may be noted, however, that railway tunnels of oval section in swelling and unstable material have been reinforced in some cases by steel ribs. This treatment, of course, was only designed to meet unexpected emergencies and is not a criterion for tunnel design, while the railway tunnels in question were of much smaller section than would be required for a double-deck tunnel.

Supplementing the double-deck oval section, the writer suggested a circular tunnel having an upper deck supported on columns and not in contact with, or in any way connected to, the tunnel wall, thus avoiding any interference with distribution of stresses in the latter, due to external pressure. In other words, there was a viaduct within the tunnel. It should be explained that neither of these ideas ever attained the dignity of a serious study as to its possibility or practicability.

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Paper No. 1435

WATER SUPPLY FOR THE CAMPS, CANTONMENTS, AND OTHER PROJECTS BUILT BY THE CONSTRUCTION DIVISION OF THE UNITED STATES ARMY*

By Dabney H. Maury†, M. Am. Soc. C. E.

With Discussion by Messrs. George W. Fuller, P. H. Norcross, Clarence Goldsmith, T. Kennard Thomson, W. P. Mason, J. Waldo Smith, E. T. Thurston, K. M. Boorman, William J. Boucher, Morris Knowles, H. F. Dunham, George A. Johnson, F. R. Harris, Maurice R. Scharff, Samuel A. Greeley, Lynn E. Perry, Asa E. Phillips, and Dabney H. Maury.

Synopsis.

This paper presents briefly some facts in connection with the organization of the Construction Division, giving credit to those civilians who were principally responsible for the selection of its personnel and for the preparation of the form of contract adopted. It then describes the general standards for the water supply systems of the cantonments and the preparation of the typical preliminary plans, giving an illustration of the manner in which time was saved by the latter.

The preparation of general plans for the distribution system, and the scheduling of materials therefor, are next described and illustrated, and studies of the various water-supply problems are outlined. The paper next sets forth the general specifications and the construction details for water supply, and illustrations of the latter are presented.

* Presented at the meeting of April 2d, 1919.

† Lt.-Col., Quartermaster Corps, Construction Division, U. S. A.

The scheduling and purchase of service pipe materials and of supplementary materials are explained, and the conditions governing the selection of the various materials used are stated.

Under the heading "Sanitation", credit is given to the Sanitary Corps of the Medical Corps for its cordial and efficient co-operation from the very beginning to date, and to the Supervising Engineers, the Engineers for the Contractors, and the Sanitary Officers assigned to the camps, for their excellent work in looking after the quality of the temporary water supplies during the construction period.

Additional sanitary precautions insisted on by the office of the Advisory Engineer for Water Supply are stated. The methods by which water-works materials for the National Guard camps were shipped to these camps in advance of plans, and in most cases in advance of the arrival of engineers, contractors, or working forces, is narrated.

The enlargement of the scope of work of the Construction Division is mentioned, and a number of working plans for water-works are presented and described.

The paper states the consumption of water in the camps and the methods adopted to check leakage and waste, and also the changes in the National Guard camps and the manner in which water supply materials were made available for these changes before such changes had been authorized.

The preparation of contracts for water supply extensions and for water service, and the rates at which water was furnished to the Government, are next treated.

Finally, the paper represents a summary of the work done and of the results accomplished in First Cost, Sanitation, and Fire Protection, and gives credit to all those who bore a responsible part in the work, whether in the office or in the field.

Organization.

Much has been written about the remarkable achievements of the Construction Division of the Army, and no man who served in that Division, in no matter how humble a capacity, can fail to be proud of having been connected with an organization so notably efficient.

It would be unfair to praise the Construction Division, however, without first giving credit to the men who were not thereafter officially

connected with it, but who in the very beginning helped to select the major part of its official personnel and to mobilize its contractors, and who collaborated with the Office of Cantonment Construction in the preparation of a form of contract absolutely different from any that had ever been used on Government work, but which has been proved to be the only form which could have produced the desired results.

The list of the men, to whose energy, ability, practical patriotism, and wise counsel the country owes so much, included, among others, Messrs. Morton C. Tuttle, Leonard Metcalf, George W. Fuller, William O. Washburne, Frederick Law Olmstead, Asa E. Phillips, and Col. William A. Starrett.

In those crucial early days in April, May, and June, 1917, after it had finally been decided that the United States should prepare to do its part in the greatest war of history; when practically no army was available and there was no place to train one; and when every additional day's delay in getting troops to France meant the loss of many more lives and vast treasure, these men spent weeks in Washington doing work of incalculable value to the country, and without other recompense than the joy of serving.

When the writer was "telegraphed" into service on May 28th, 1917, the work of these men had begun to show results. The Office of Cantonment Construction, starting with a personnel of 10, already numbered 40 or 50, and the men who were lucky enough to be selected were racing in from all parts of the country to take up their duties. In October, 1917, the number had increased to 700; in March, 1918, to 1 400; and when the Armistice was declared, on November 11th, 1918, the force in the Washington office alone numbered 2 800, and about 25 000 more, scattered all over the United States and its insular possessions, were reporting to Gen. Marshall, Chief of Construction Division, which number did not include any of the 200 000 laborers or other employees of the contractors.

The day before the writer's arrival in Washington, Clarence Goldsmith, M. Am. Soc. C. E., Hydraulic Engineer of the National Board of Fire Underwriters, had reported to the Officer in Charge of Cantonment Construction. The services of Mr. Goldsmith had been generously donated to the Government by the National Board of Fire Underwriters up to April 6th, 1918, when he was commissioned a Major in the Quartermaster Corps. From May 28th to date he has been Prin-

cipal Assistant to the Advisory Engineer for Water Supply. His loyalty, untiring energy, and exceptional ability, as a designer of water-works and fire protection systems, have saved millions for the Government.

During the period of construction of the Cantonments and National Guard Camps, no further increase was made in the personnel of the Office of the Advisory Engineer for Water Supply, the services of such assistant engineers, computers, draftsmen, or stenographers as were required being supplied from the general offices of the Construction Division. In the latter part of 1917, the number and diversity of the problems submitted to the office necessitated the organization of an additional and independent force. The members of this force were, without exception, active, industrious, and faithful, and to every one of them the writer desires to give credit for work well done, and to express his grateful appreciation of their loyal co-operation.

General Standards.

In all water-works designed by this office, the first consideration has always been the sanitary quality of the supply, of which more will be said later. This being assured, the first point to be determined in the actual design of the water-works system was the quantity of water required. This was basic, and entered into every part of the design.

It was necessary to provide a supply sufficient, in the practical operation of the cantonment, for every reasonable requirement of the troops, for domestic use, as well as for proper fire protection. The restrictions and limitations included the following: the limited appropriations; the enormous prices of material and labor; the difficulty of obtaining on short notice as much of either as would be required for the simultaneous construction of sixteen camps to house about 40 000 troops each; the necessity of avoiding, in so far as possible, the use of steel or iron, both of which were urgently needed for ships, guns, and munitions; and, above all, the absolute necessity for haste.

It was early recognized that some leakage and waste would be unavoidable; at the same time it was believed that the concentration of most of the water fixtures in lavatories, where from forty to seventy outlets could be inspected almost at a glance, and the fact that, under military discipline, proper administrative rules could be enforced,

would combine to make it an easy matter to reduce leakage and waste to relatively small quantities.

More consideration was given to the determination of the proper allowance of water than to any other question, and Leonard Metcalf, and George W. Fuller, Members, Am. Soc. C. E., collaborated with this office in the consideration of this important and fundamental subject.

After the quantity of water required for domestic consumption had been determined, the question of fire protection was studied, and the requisite quantity and the pressure at which this quantity should be available were also determined.

Typical Preliminary Plans.

The broad and general engineering standards which should govern the construction of the water systems to supply the camps having been determined, the next most important problem was the preparation of such plans for the individual systems as would require the minimum time for construction, utilizing such material as would not hamper the progress of shipbuilding and other vital war industries, and at the same time would furnish safe, adequate, and reliable service.

Typical plans of the building layouts of the several units of a standard divisional cantonment were prepared by the Camp Planning Section and transmitted to this office, as were also typical plans showing various methods in which these several units could be grouped on the ground. It was manifest at the outset that no particular arrangement of units could be adhered to for all the camps, as the grouping would necessarily have to be governed in each case by the topography of the camp site. It was possible, however, in almost every case, to adhere strictly to the group plan of buildings in each of the several units, and this permitted the preparation of standard plans showing the location of mains, and the location and sizes of the service pipes in each unit.

Fig. 1 is a typical plan of the mains and service pipes in a Regimental Unit. The lengths and sizes of the mains, and their location outside the limits of the individual units, were governed in each case by the topography, which dictated the way in which the units were strung together, and also by the point at which the water supply was delivered to the camp.

Plate VIII shows several typical arrangements of water distribution mains, corresponding to various groupings of the units in a Divisional Cantonment. In one case the units are strung out in a single line, with the supply coming in at one end; in another, with the units strung out as before, the supply comes in at the center; and, in the third case, the units are grouped in horse-shoe form. This latter arrangement was the ideal one, as it gave an opportunity to loop the mains across the ends of the horse-shoe, which greatly reduced the friction losses and enabled smaller mains to be used.

In the preparation of these typical plans, studies were made to determine in each case the economic location and sizes of the mains, gates, and fire hydrants, so that, by the time the first rough plan showing the approximate actual grouping of the units on the ground in any particular camp was submitted to this office, it might be possible, with the minimum of delay, to specify offhand the sizes and the approximate lengths of mains required and to place orders for the bulk of the water-works materials.

As an instance of the way in which time was saved by this method, the case of Camp Lewis may be cited. It had been decided to build a cantonment near American Lake, Washington, but two local factions had been unable to agree on the proper location, and Maj., afterward Col., Peter Junkersfeld,* of the Building Division of the Construction Division, was sent out to settle the matter and fix the final location of the camp. On the day after his arrival, he sent a telegram which read as follows:

"Refer Government Maneuver Map No. 69. Follow railroad southwesterly from American Lake to Dupont Station. Draw line easterly from this station to point midway between Spray Lake and Spray Farm. This line is the center axis of camp, which is in typical horse-shoe plan with open end eastward. Source of water supply, wells and springs between American Lake and Sequalitchew Lake. Site for reservoir, Hill No. 320; actual elevation, 390. Engineering Division wire sizes and lengths of water mains required."

This telegram arrived at the Washington office just before midnight, and within two hours the Engineering Division wired the information required, which information enabled the pipe for the distribution mains of the camp to be ordered immediately from the nearest Pacific Coast fir pipe manufacturers. A few hours thus sufficed to settle a basic question in connection with a cantonment more

* Now M. Am. Soc. C. E.

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
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MAURY ON
WATER SUPPLIES FOR
CANTONMENTS, CAMPS, ETC.

CANTONMENT GROUNDS
TYPICAL WATER AND SEWERAGE PLAN
GENERAL PLAN

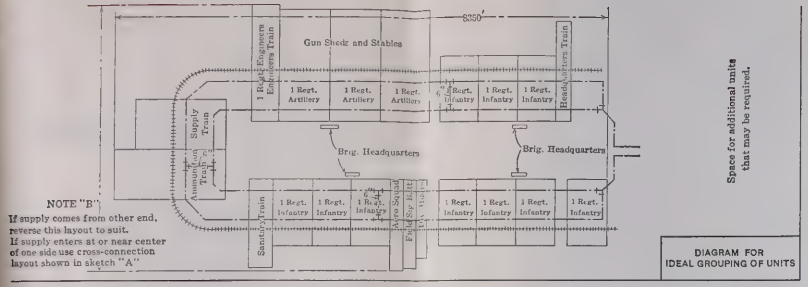
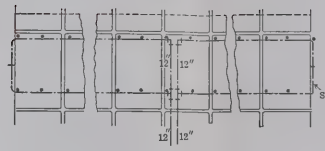


DIAGRAM FOR IDEAL GROUPING OF UNITS

NOTE "B"
If supply comes from other end, reverse this layout to suit.
If supply enters at or near center of one side use cross-connection layout shown in sketch "A"

NOTE "A"
For center feed use layout and pipe sizes as per sketch "A". All 6" pipe lines to be the same for center feed as for end feed, except that the central 6" cross-connection shown on end feed plan is replaced in the center feed plan by the two 12" cross-connections shown on sketch "A".

SKETCH "A"



GENERAL NOTE
The number and arrangement of units will, of course, vary for each cantonment, and it is likely that in no case will they correspond closely to this plan.
It is important, however, that the location of water mains and hydrants, with regard to each unit in each cantonment, should follow closely the layout shown hereon. No matter how much it may be necessary to alter the positions of the respective units, or to curve the mains between them, the mains should run straight across each unit, and in the positions shown on this plan.

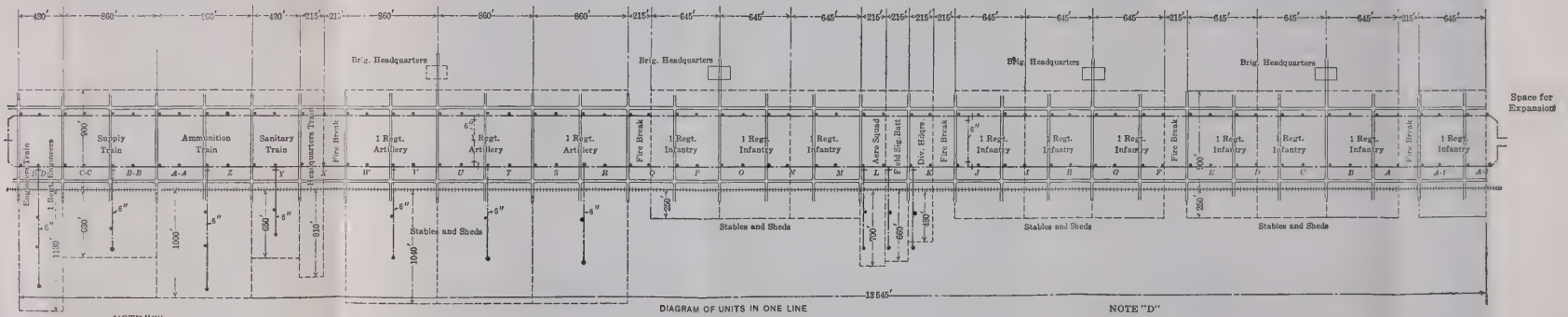


DIAGRAM OF UNITS IN ONE LINE WITH END FEED FOR WATER SUPPLY

NOTE "C"
WATER PIPES
For layout of water pipes for each unit, see typical water and sewerage plans for each unit.
For construction details of water pipes, see construction detail sheets.

LEGEND
Water Pipe
Hydrants
Gates

Scales
1000 500 0 1000 2000 3000
¼ Mile ½ Mile

NOTE "D"
SEWERS
For layout of sewers for each unit, see typical water and sewerage plans for each unit.
Trunk sewers are to be planned by engineer in charge of each cantonment to suit the topography and other conditions of that particular site.

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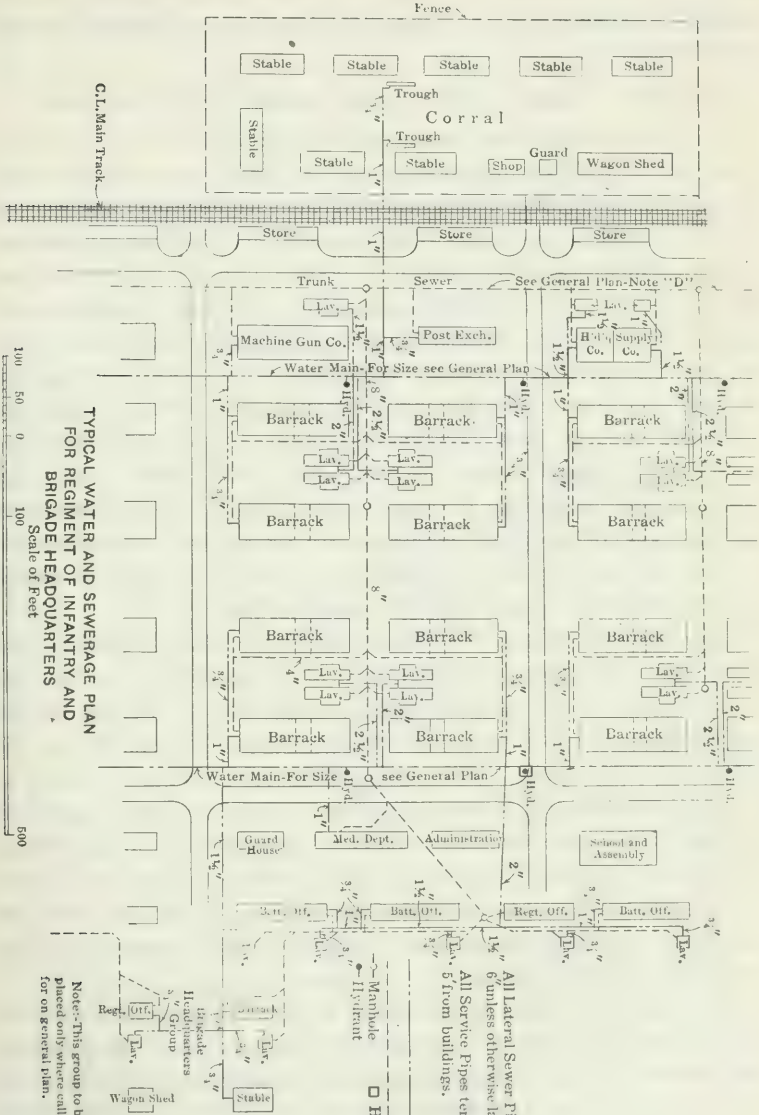


TYPICAL WATER AND SEWERAGE PLAN FOR REGIMENT OF INFANTRY AND BRIGADE HEADQUARTERS

FIG. 1.

than 3 000 miles away. Although the location of this camp was not definitely decided until after June 28th, the camp was ready for troops on September 1st.

Plans of Distribution System and Schedules of Materials.

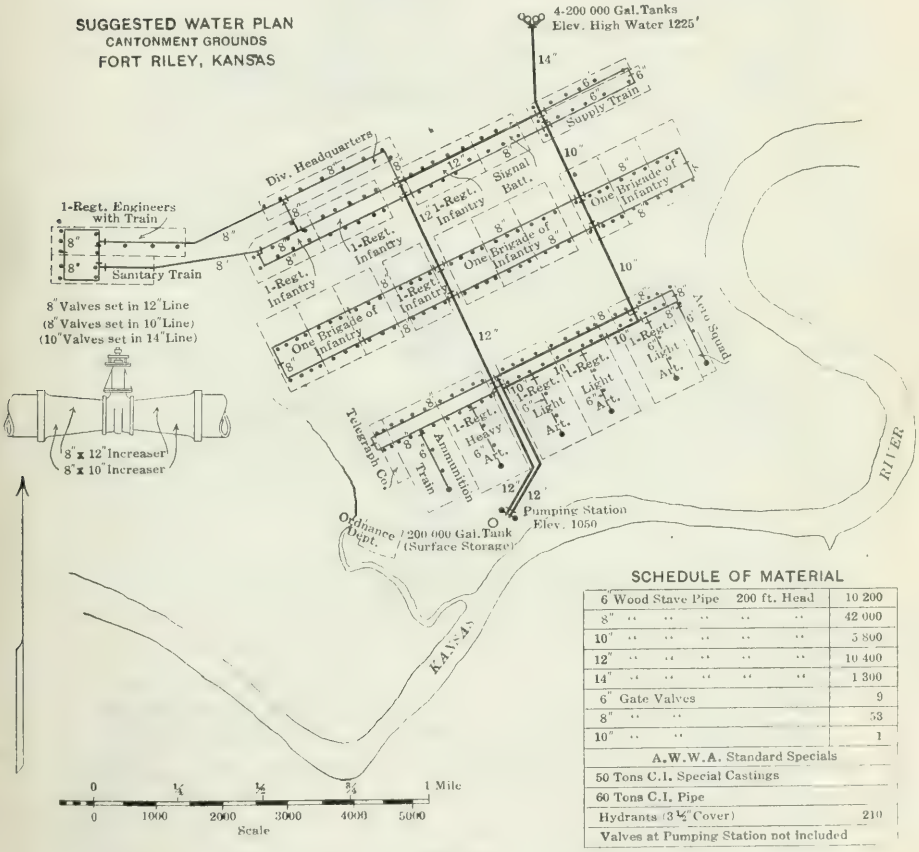
The City Planners and Engineers on the ground at each camp sent in as fast as possible topographic maps of each camp site, on which were shown the general arrangement of the several units; and as soon as this general arrangement was approved by the Camp Planning Section, a blue print was furnished to this office. The locations and sizes of the water mains, fire hydrants, valves, and other accessories were immediately drawn on these blue prints with red pencil; from these red pencil marks the necessary materials were scheduled, and the schedule was sent to the Materials Division for purchase. The blue print with the red marks on it was then sent back to the Drafting Room of the Camp Planning Section, where it was traced as a water-works drawing, and given its proper number. Blue prints from these tracings were forwarded to the camps, but the method of handling the work in this office actually enabled most of the water supply materials to be shipped to the camp before the plans arrived.

Fig. 2 shows the water distribution system for Camp Funston. The supply was from wells, low-lift pumps at which delivered water into surface storage at the pumping station, whence it was pumped through the distribution mains and into elevated storage on the hills on the other side of the camp. This arrangement gave an ideal system, as the elevated reservoirs furnished a second source of supply, and their location on the opposite side of the camp greatly reduced the friction losses in the mains. It will also be noted that there are duplicate supply mains from the pumping station as well as from the reservoirs; that the distribution mains are looped everywhere; and that there are no dead ends.

Fig. 3 shows the water-works layout for Camp Grant, which embodies many of the same features as that for Camp Funston, the principal difference being that at Camp Funston the near-by hills enabled wood-stave tanks to be used for storage, and at Camp Grant the flat topography compelled the use of the more expensive steel tank on a steel tower.

Fig. 4 shows the arrangement of mains and service pipes for a typical base hospital unit. The presence of the corridors connecting

all the buildings made it difficult to furnish effective fire protection to the hospitals; first, because they eliminated fire-breaks and offered a fine chance for a fire to spread from one building to another; and, second, because they blocked access for the fire fighters to the interior



SCHEDULE OF MATERIAL

6" Wood Stave Pipe 200 ft. Head	10 200
8" " " " " "	42 000
10" " " " " "	5 800
12" " " " " "	10 400
14" " " " " "	1 300
6" Gate Valves	9
8" "	53
10" "	1
A.W.W.A. Standard Specials	
50 Tons C.I. Special Castings	
60 Tons C.I. Pipe	
Hydrants (3½' Cover)	210
Valves at Pumping Station not included	

FIG. 2.

courts between the corridors and the buildings. To offset these disadvantages, as much as possible, all hospital groups were given duplicate supply mains, the distribution mains were very thoroughly looped and gated, the hydrant spacing was made unusually close, and, wherever possible, openings were left under the corridors at strategic points.

Studies of Water-Supply Problems.

The foregoing describes more particularly the method of procedure in regard to the distribution system, which naturally formed the larger part of the water-works construction. The selection of the source of

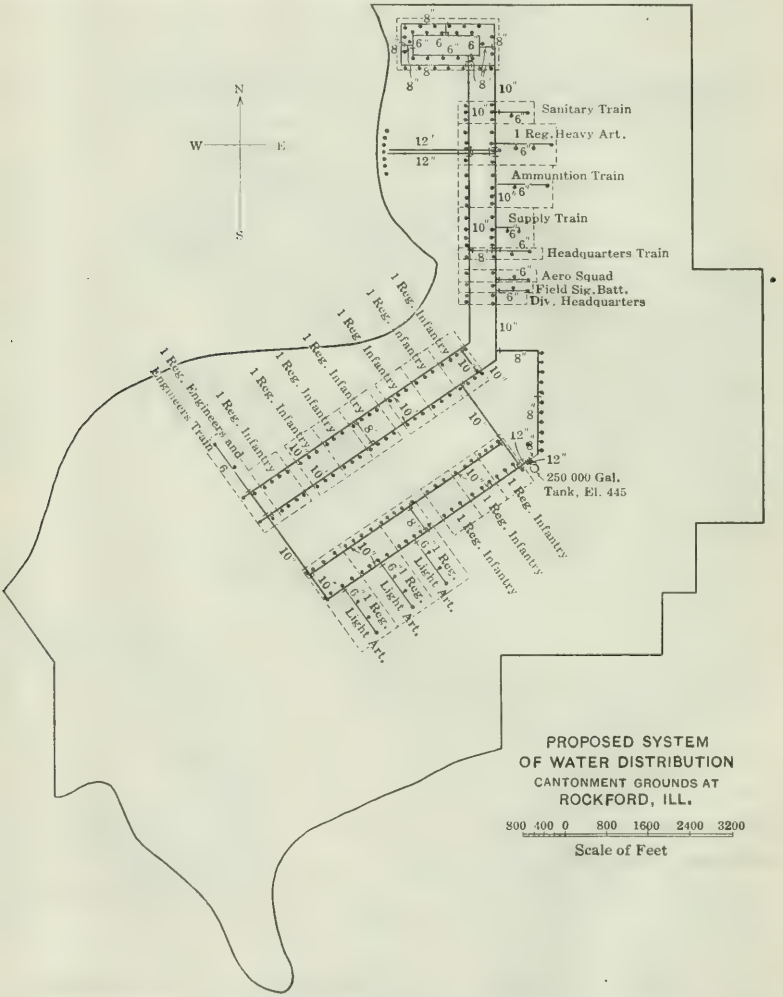
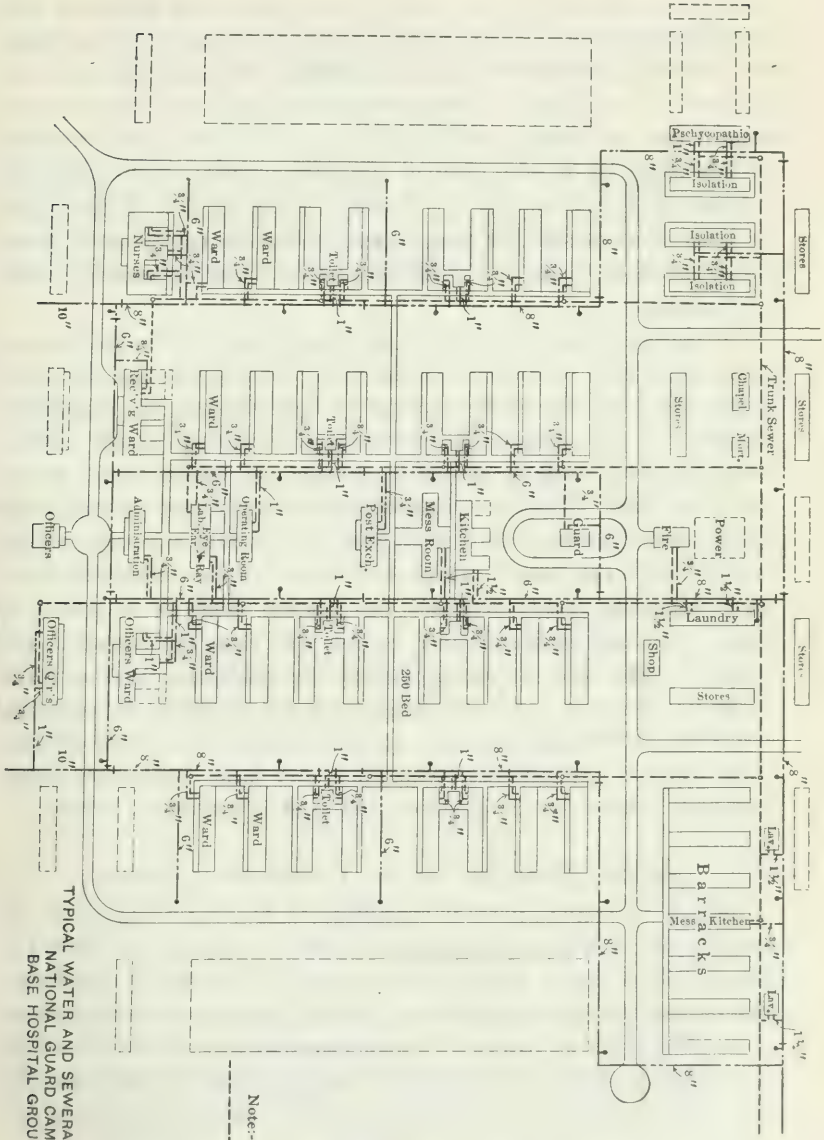


Fig. 3.

supply, the determination of the method of bringing it to the camp, and the planning of pumping stations, purification works, or reservoirs, if any or all of these were required, were usually governed largely by



TYPICAL WATER AND SEWERAGE PLAN
NATIONAL GUARD CAMP
BASE HOSPITAL GROUP

FIG. 4.

information conveyed in the reports of the supervising engineers from the camps themselves. Each of these problems was dealt with as fast as the necessary information became available. In some instances, the information was already at hand, either as a result of previous knowledge of the local water supply conditions on the part of the engineers in this office, or in the form of previous reports. Quite often the excellent reports, issued by the National Board of Fire Underwriters, on cities from which the camp supply might be taken, made immediately available most of the information required in regard to such city supplies.

In the beginning, of course, it was impossible, with no data and no time available, to send detailed plans from Washington, and the efforts of the office were necessarily confined to impressing on the supervising engineers at the camps the necessity of adhering to the general standards established by the office, leaving the details to be worked out in the field. These men included many of the very best engineers in the United States; but it was quite natural that any engineer not familiar with the reasons which had controlled the establishment of these standards, should approach the problem much as he would the design of a metropolitan water system, and plan for capacities sufficient to take care of normal growths in population over a period of 20 or 25 years. As opportunity offered, conferences were held with these supervising engineers, at which the reasons for the adoption of the various standards were explained, and these conferences almost invariably cleared up the situation and produced excellent results.

General Specifications and Construction Details.

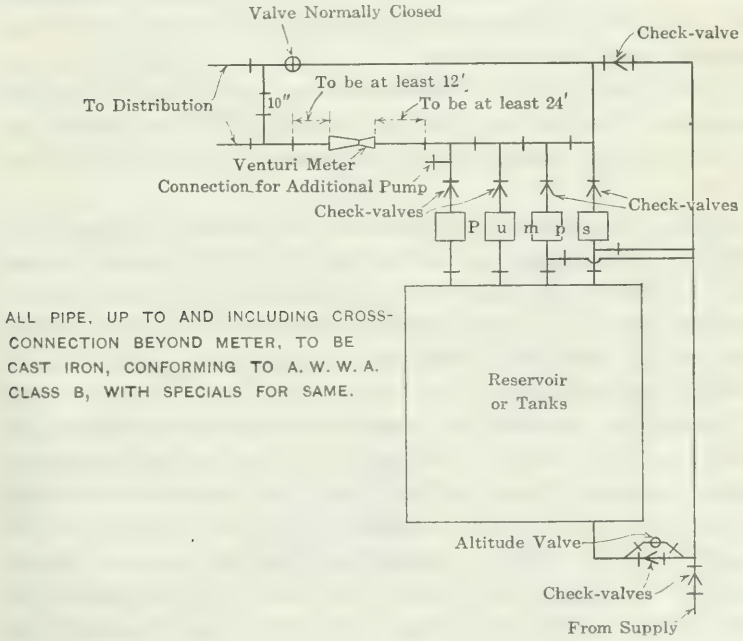
In the meantime, general specifications for water supply to the cantonments, and typical construction details showing the method of setting valves, valve boxes, and hydrants, of connecting wood-stave pipe to cast-iron pipe or specials, of making connections between mains, of making bends in mains, of connecting service pipes to mains, the arrangements of pipe connections to pumping stations and reservoirs, etc., were prepared and sent out, as well as a schedule of service pipe materials necessary for each individual unit in a cantonment.

Fig. 5 is a diagrammatic layout used in planning the piping connections at pumping stations and reservoirs. The duplication of suction and discharge mains, and the location of gate-valves and check-

TYPICAL ARRANGEMENT FOR PUMPING STATION

CONSTRUCTION DETAILS
FOR

TYPICAL LAYOUT OF PUMPS, RESERVOIR, TANKS, METER AND PIPE CONNECTIONS, FOR WATER SYSTEMS IN WHICH SUPPLY IS DELIVERED INTO STORAGE AND HAS TO BE PUMPED THENCE INTO THE DISTRIBUTION MAINS, AND WHERE THERE IS NO ELEVATED STORAGE ON DISTRIBUTION MAINS. WHERE THERE IS SUCH ELEVATED STORAGE, THE NUMBER OF PUMPING UNITS MAY BE REDUCED.



WHERE PUMPS DO NOT TAKE SUCTION UNDER HEAD, PROVIDE SUITABLE PRIMING APPLIANCES IN DUPLICATE.

WHERE EXCESSIVE PRESSURES MIGHT OTHERWISE COME ON MAINS, AS THE RESULT OF SHUTTING OFF DRAFT WHILE PUMPS ARE RUNNING AT HIGH RATES, SUITABLE RELIEF VALVES SHOULD BE PROVIDED AT PUMPING STATION.

FIG. 5.

valves, make it impossible for the failure of any one valve to put more than two pumping units out of commission, or for the failure of any pipe or special casting to put more than one unit out of commission. Attention is called to the by-pass around the Venturi meter and to the arrangement of the altitude valve on the line supplying the reservoir.

In Fig. 6 are three construction details, the one in the upper left corner showing a bend in a wood-stave pipe line, with two 45° L's and a length of cast-iron pipe on each side to prevent blow-off. The lower left detail shows a typical connection between existing and new wood-stave mains, by which the use of a cross is avoided. Crosses are more often defective than tees, and the failure of a cross puts all four connecting lines out of service.

The detail on the right illustrates the method of connecting altitude valves. The full size of the outlet check gives free outflow, and the reduced size of altitude valve saves money. The bends facilitate repairs.

In Fig. 7 the upper left detail shows the method of connecting a hydrant to a wood-stave main, and the short 4-in. hydrant branch used. The lower left detail shows the connection of branch lines to wood-stave mains; the upper right shows the connection of a hydrant to the end of a 6-in. wood-stave main; and the lower right shows the method of protecting valves and valve boxes from injury by traffic.

Fig. 8 shows the lead joint used for connecting wood-stave pipe to cast-iron pipe, valves, or fittings, and describes the method of making the joint. When the first orders were placed for wood-stave pipe the office was definitely informed that it was common practice to lead the tenons of the wood pipe into standard cast-iron hubs in the same way that cast-iron spigots are leaded in; but reports soon began to come in from the camps to the effect that this was by no means an easy thing to do. A great many methods were suggested for making these joints, but by far the best of all was that shown on Fig. 8, the idea for which was suggested at about the same time by Mr. Ramsey, Water-Works Engineer for Morris Knowles, Incorporated, Supervising Engineers for Camp Meade, and by an engineer for a camp in the Middle West. The success of the method depended largely on having all the tenons made small enough to provide proper lead space, and on having the diameter of the tenons accurately

DETAILS.
90° CURVE WITH WOOD STAVE PIPE,
EXISTING AND NEW WOOD STAVE MAINS,
STANDARD SETTING FOR ALTITUDE VALVE.

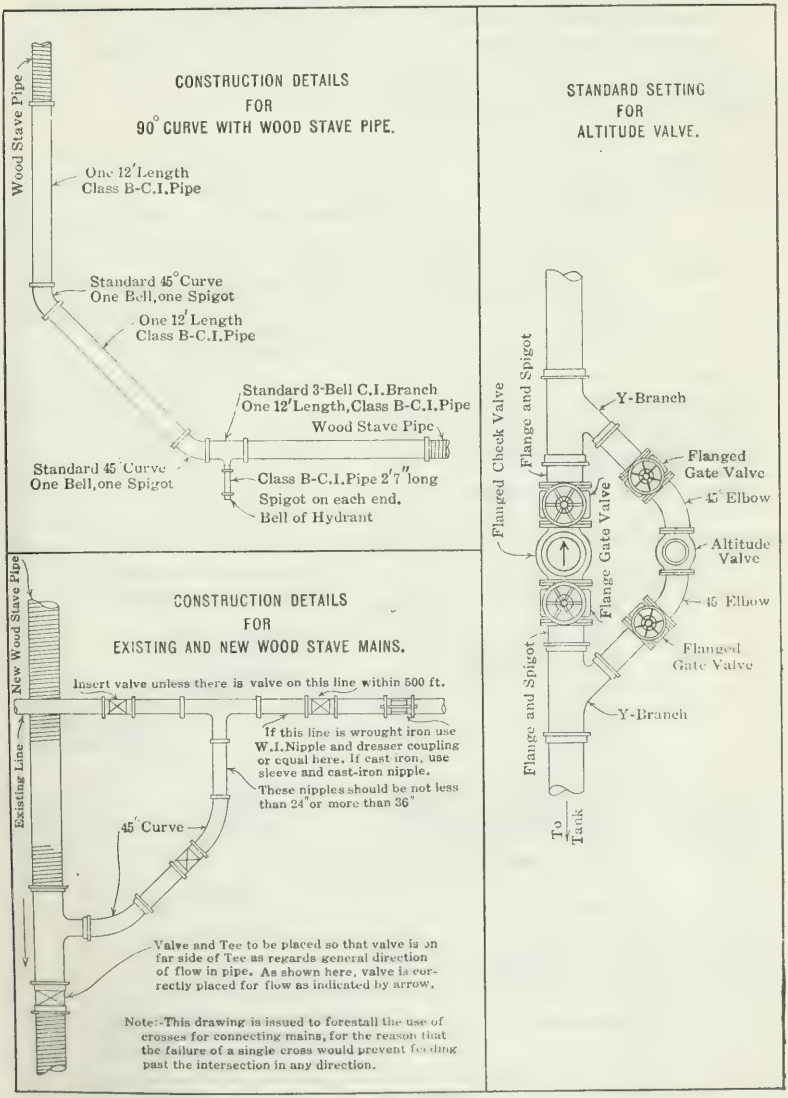
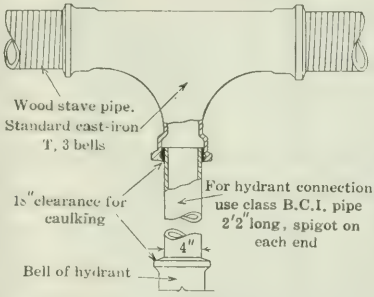

CONSTRUCTION DETAILS
FOR
90° CURVE WITH WOOD STAVE PIPE.
Wood Stave Pipe
One 12′ Length
Class B-C.I.Pipe
Standard 45° Curve
One Bell, one Spigot
One 12′ Length
Class B-C.I.Pipe
Standard 3-Bell C.I.Branch
One 12′ Length, Class B-C.I.Pipe
Wood Stave Pipe
Standard 45° Curve
One Bell, one Spigot
Class B-C.I.Pipe 2′7″ long
Spigot on each end.
Bell of Hydrant
CONSTRUCTION DETAILS
FOR
EXISTING AND NEW WOOD STAVE MAINS.
New Wood Stave Pipe
Existing Line
Insert valve unless there is valve on this line within 500 ft.
If this line is wrought iron use W.I.Nipple and dresser coupling or equal here. If cast-iron, use sleeve and cast-iron nipple.
These nipples should be not less than 24" or more than 36"
45° Curve
Valve and Tee to be placed so that valve is on far side of Tee as regards general direction of flow in pipe. As shown here, valve is correctly placed for flow as indicated by arrow.
Note:-This drawing is issued to forestall the use of crosses for connecting mains, for the reason that the failure of a single cross would prevent feeding past the intersection in any direction.
STANDARD SETTING
FOR
ALTITUDE VALVE.
Flanged Check Valve
Flange and Spigot
Y-Branch
Flanged Gate Valve
45° Elbow
Flange Gate Valve
Altitude Valve
45° Elbow
Flanged Gate Valve
Flange and Spigot
Y-Branch
To Tank

FIG. 6.

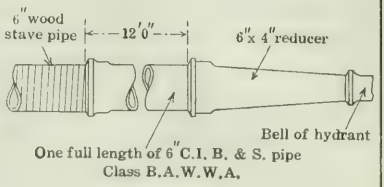
DETAILS
WOOD STAVE PIPE,
BRANCH LINE, VALVES, AND VALVE BOXES.

DETAILS FOR CONNECTING FIRE HYDRANT TO SIDE OF MAIN.

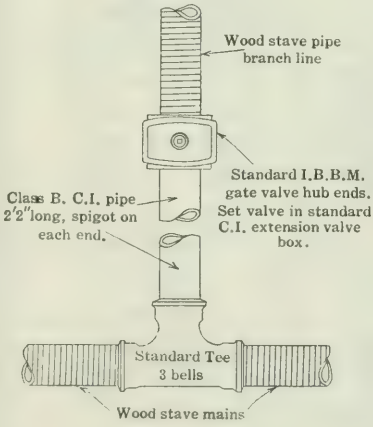

Wood stave pipe.
Standard cast-iron T, 3 bells
1s"clearance for caulking
For hydrant connection use class B.C.I. pipe 2′2″long, spigot on each end
4"
Bell of hydrant

Hydrant to be of reputable make, to have valve opening at least 4″diam., and to discharge 500 gal. per min., with friction loss of not more than 3 lb. Length of barrel to suit depth of cover required at cantonment.
If branch is longer than 2′2″use 6″pipe.

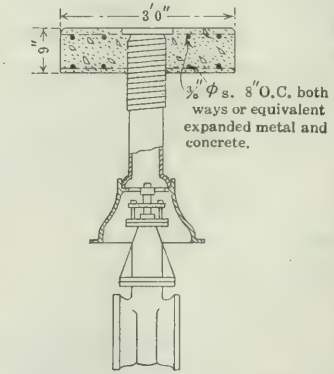
DETAILS FOR CONNECTING FIRE HYDRANT TO END OF MAIN.



DETAILS FOR CONNECTING BRANCH LINES TO MAIN.


Wood stave pipe branch line
Standard I.B.B.M. gate valve hub ends. Set valve in standard C.I. extension valve box.
Class B. C.I. pipe 2′2″long, spigot on each end.
Standard Tee 3 bells
Wood stave mains

DETAILS FOR PROTECTING VALVES AND VALVE BOXES.



In setting valve box, leave at least 4″ between valve box and gate at nearest point. Cast around top of each valve box a 1: 2½: 5 concrete disk 3′0″diam. and 9″thick, as shown.

FIG. 7.

LEAD JOINT FOR CONNECTING REDWOOD OR FIR STAVE PIPE TO CAST-IRON PIPE, VALVES, OR FITTINGS.

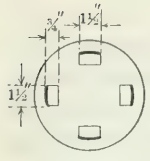


FIG. *a*

BOTTOM VIEW OF PLUG SHOWING RECESSES IN ¾" EXTENSION TO ACCOMMODATE THE LUGS ON VALVE SEATS.

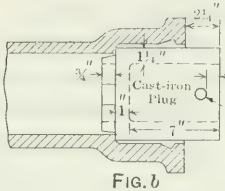


FIG. *b*

DIAGRAM OF CAST-IRON BELL BEFORE POURING LEAD JOINT, SHOWING PLUG IN PLACE.

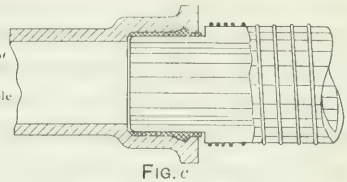


FIG. *c*

DIAGRAM SHOWING LEAD JOINT BETWEEN WOOD STAVE PIPE AND CAST-IRON PIPE, OR FITTINGS.

TO MAKE JOINT:

INSERT PLUG IN BELL AS SHOWN IN FIG. *b*

POUR LEAD JOINT AND CAULK LIGHTLY, TO ENSURE TIGHT JOINT BETWEEN LEAD AND INSIDE OF BELL.

REMOVE PLUG, LEAVING LEAD IN BELL.

DRIVE TENON OF WOOD STAVE PIPE INTO BELL, AS SHOWN IN FIG. *c*

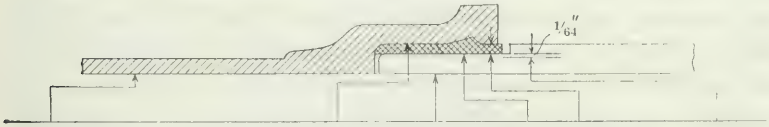
CAULK LEAD JOINT LIGHTLY IF NECESSARY.

TENON MAY BE BRUSHED WITH THIN SOFT SOAP JUST BEFORE INSERTING IN JOINT.

PLUG IS TO BE SMOOTHLY TURNED WITH TAPER OF 1/16" IN 4". OUTSIDE DIAMETER OF PLUG AT FACE OF BELL IS TO BE 1/32" LESS THAN OUTSIDE DIAMETER OF TENON OF WOOD STAVE PIPE. OUTSIDE DIAMETER OF PLUG AT BOTTOM OF BELL IS TO BE 1/16" LESS THAN OUTSIDE DIAMETER OF PLUG AT FACE OF BELL.

WHEREVER IT IS POSSIBLE TO DO SO, IT WILL BE FOUND MORE CONVENIENT TO POUR THE LEAD AND REMOVE PLUG BEFORE PLACING THE CAST-IRON PIPE OR FITTING IN THE TRENCH.

TABLE OF SIZES FOR CAST-IRON PIPE, REDWOOD OR FIR STAVE PIPE AND FITTINGS. AMERICAN WATER WORKS ASSOCIATION STANDARD.



CAST-IRON PIPE AND FITTINGS.				WOOD STAVE PIPE.			
Nominal Diameter	Class		Minimum I. D. Bell	Inside Diam. Wood Pipe	Outside Diam. Tenon	Min. Thick's of Lead	O.D. Plug at Face of Bell
	Pipe	Fittings					
4"	*BCD*	*BCD*	5.70"	4"	5 7/16"	0.145"	5 13/32"
6"	"	"	7.80"	6"	7 1/2"	0.165"	7 16/32"
8"	*CD*	*ABCD*	10.00"	8"	9 3/8"	0.140"	9 23/32"
10"	"	"	12.10"	10"	11 5/8"	0.255"	11 19/32"
12"	*ABCD*	"	14.00"	12"	13 3/4"	0.140"	13 23/32"
14"	"	"	16.10"	14"	15 3/4"	0.175"	15 23/32"
16"	"	"	18.40"	16"	18"	0.220"	17 31/32"
18"	"	"	20.50"	18"	20"	0.270"	19 31/32"
20"	"	"	22.60"	20"	22"	0.320"	21 31/32"
24"	"	"	26.80"	24"	26"	0.420"	25 31/32"

FIG. 8.

standardized. The details of the plug and the dimensions of the tenons were worked out by the office with the company which furnished most of the wood-stave pipe, and the other pipe-makers soon agreed to make their tenons of the same dimensions.

Fig. 9 shows, on the left, the method of frost-proofing service pipes under the floors of buildings, the intention being to provide a double air space around the pipe. It was sometimes difficult to prevent the construction forces from defeating the purpose of the device by filling these air spaces with saw-dust or other material, which, when wet, destroyed the non-conducting value of the air space. The details on the right of Fig. 9 show two methods of connecting service pipes to wood-stave mains.

Service Pipes and Fittings.

An estimate of quantities of the service pipe and fittings required for the National Army Cantonments was prepared as per Table 1 (Plate IX), which shows in its upper half the service pipe material required for each unit of each type in a divisional cantonment. The total service pipe material required for each of the sixteen cantonments is shown in the lower half of Table 1, and was readily determined when the number and type of units in any cantonment were once known.

Supplementary Schedules of Materials.

After detailed plans, based on the general plans and specifications, were worked up by the supervising engineers in the field, they would submit, as fast as possible, additional schedules of materials which might be required to complete the work, and these schedules were forwarded to the Materials Division for the purchase of the material, as soon as they had been approved by this office. To expedite the work still further, blanket authority was given in certain cases to the construction forces to purchase, without further reference to Washington, limited quantities of such specified classes of materials as could best be purchased in that manner, instead of through the usual channels.

Conditions Governing Selection of Materials and Equipment.

It may be interesting to note at this point that it was usually possible to deliver water, from the source of supply which had been selected, into storage at the camp. This storage was generally at such

TABLE 1.—Estimate of Quantities of Pipe and Fittings for Water Distribution of National Army Cantonments.

	Kind	Cast Iron 90° Elbow.				C. I. 45° Elbow.		Black Cast-Iron Tees.																Bushings.				Reducing Couplings.				Corporation Cock.				Gal. Iron Pipe.			
	Size, in inches	2	1½	1	¾	1	¾	2 × 1½ × 2	2 × 1½ × 1½	2 × 1½ × 1	2 × 1½ × ¾	2 × 1¼ × 2	2 × 1¼ × 1½	2 × 1¼ × 1	2 × 1¼ × ¾	2 × 1 × 2	2 × 1 × 1½	2 × 1 × 1	2 × 1 × ¾	1½ × 1 × 1	1½ × 1 × ¾	1½ × ¾ × ¾	1¼ × ¾ × ¾	2 × 1¼	2 × 1	1½ × 1	1 × ¾	2 × 1¼	1½ × 1	1 × ¾	¾ × ½	1¼	1	¾	½	2	1½	1	¾
A	Division headquarters, field signal and aero squad.	3	2	2	6				3		1	3	6					1	4					3	1	6	11	3	1	6	6	220	410	830	1 070
B	Brigade headquarters				5								4	1					2															90	20	270
C	Regiment of infantry	6			12	1		1	2	7			15			1	1		9	8	2	3	1	8	9	8	1	1	350	640	870	1 650
D	Regiment of light artillery	4	2		10		2	1	4	1		4	6		1	1		2	5	4	2	2	17	4	2	3	17	560	1 065	940	2 240
E	Regiment of heavy artillery	4	1	2	14				3			7	4					2	4	3	3	5	11	3	3	5	11	590	710	645	1 005
F	Regiment of engineers with train	4	3		10			1	6	3	1	4	7			2		2	6	6		4	9	6		4	11	750	280	475	1 070
G	Pack squad	1			2					1			1						1	1		2		1		2			150		220	295
H	Headquarters train		3	1	4					1			2						1		1	3	7		1	3	7			300	360	665
I	Ammunition train	2	2	1	8			1	2	2	1		3						2	3	2	2	16	3	2	2	16	430	360	283	1 490
J	Supply train			2	2			1		2		1	1						6	5	1	3	2	4	1	3	2	4		160	280	710
K	Sanitary train	4			5		4		4				6		3					5	4		8	11	4		8	5	600		630	1 660
L	Pack-mule company		1		4								1							1		1	1	1	1	1	1		120	190	140
M	Telegraph company			1	6				1				2							3	1		8	1		4		150		535	410
N	Hospital unit			18	48		4			2	1	1	6	17					14	19		8	41	9		8	41	9		680	2 800	4 000
1	Ayer, Mass.	88	15	9	221	10	10	13	31	112	8	1	27	217	4	6	5	10	30	151	114	31	123	120	114	31	123	120	18 100	11 375	14 250	32 620
2	Yaphank, N. Y.	88	15	9	225	10	10	13	31	112	8	1	27	215	4	6	5	10	30	146	113	31	119	120	113	31	119	120	17 950	11 375	13 715	32 310
3	Wrightstown, N. J.	90	15	10	243	10	10	13	31	111	8	1	34	219	4	6	5	10	32	152	116	35	125	132	116	35	125	132	17 950	12 105	14 550	38 355
4	Annapolis Junc., Md.	88	14	8	221	10	10	13	31	98	8	1	27	214	4	6	5	10	30	147	118	30	118	119	118	30	118	119	17 950	11 255	13 535	32 070
5	Petersburg, Va.	102	16	10	265	12	10	15	33	115	8	1	34	249	4	6	5	12	34	170	132	38	140	133	132	38	140	133	19 050	16 165	16 445	36 515
6	Columbia, S. C.	94	15	9	243	11	10	13	33	106	8	1	27	262	4	6	5	11	31	164	122	33	131	121	122	33	131	121	19 450	11 915	15 120	34 270
7	Atlanta, Ga.	93	14	8	231	11	10	14	33	104	8	1	27	228	4	6	5	11	31	155	120	32	124	120	120	32	124	120	19 150	11 795	14 175	33 430
8	Chillicothe, Ohio	87	14	8	225	10	10	13	31	98	8	1	27	215	4	6	5	10	28	149	113	30	120	119	113	30	120	119	17 950	11 255	13 940	32 390
9	Louisville, Ky.	97	13	6	229	13	10	16	35	114	8	1	20	259	4	6	5	12	30	160	125	31	127	110	125	31	121	110	20 000	11 625	14 400	34 075
10	Battle Creek, Mich.	85	13	6	205	10	10	13	31	94	8	1	20	209	4	6	5	10	28	142	109	27	111	108	109	27	111	108	17 300	10 545	12 660	30 775
11	Rockford, Ill.	87	14	8	219	10	10	13	31	97	8	1	27	213	4	6	5	10	30	146	112	30	116	119	112	30	116	119	17 800	11 255	13 905	31 780
12	Little Rock, Ark.	91	13	6	217	11	10	14	33	101	8	1	20	224	4	6	5	11	29	151	117	29	119	109	117	29	119	109	18 650	11 085	13 580	32 425
13	Des Moines, Iowa	101	16	9	263	12	10	15	35	116	8	1	34	249	4	6	5	12	34	171	132	37	141	132	132	37	141	132	21 150	13 045	16 225	36 495
14	Fort Riley, Kans.	105	14	8	261	13	10	16	37	119	8	1	27	260	4	6	5	13	31	176	137	33	164	132	137	33	164	132	23 000	12 875	16 450	37 140
15	Fort Sam Houston, Texas	100	14	6	245	12	10	15	35	112	8	1	27	244	4	6	5	12	32	165	129	34	134	121	129	34	134	121	20 650	12 335	15 285	35 370
16	American Lake, Wash.	106	17	12	291	13	10	16	38	122	8	1	53	258	4	6	5	12	36	183	138	42	153	145	138	42	152	145	22 100	13 995	17 910	38 650
	Total quantities required	1 502	232	132	3 814	176	160	221	528	1 730	128	16	458	3 715	64	66	80	176	496	2 530	1 942	523	2 064	1 950	1 942	523	2 064	1 950	307 200	191 000	235 365	543 570

Notes.—All fittings black cast iron. All pipe standard galvanized wrought steel.

STANDARD DETAILS OF PROTECTION FOR FIRE HYDRANTS, HYDRANT WRENCH, CONNECTING SERVICE PIPE TO WOOD MAIN, FROST-PROOFING FOR VERTICAL PORTION OF SERVICE PIPE BETWEEN GROUND AND FLOOR OF BUILDING.

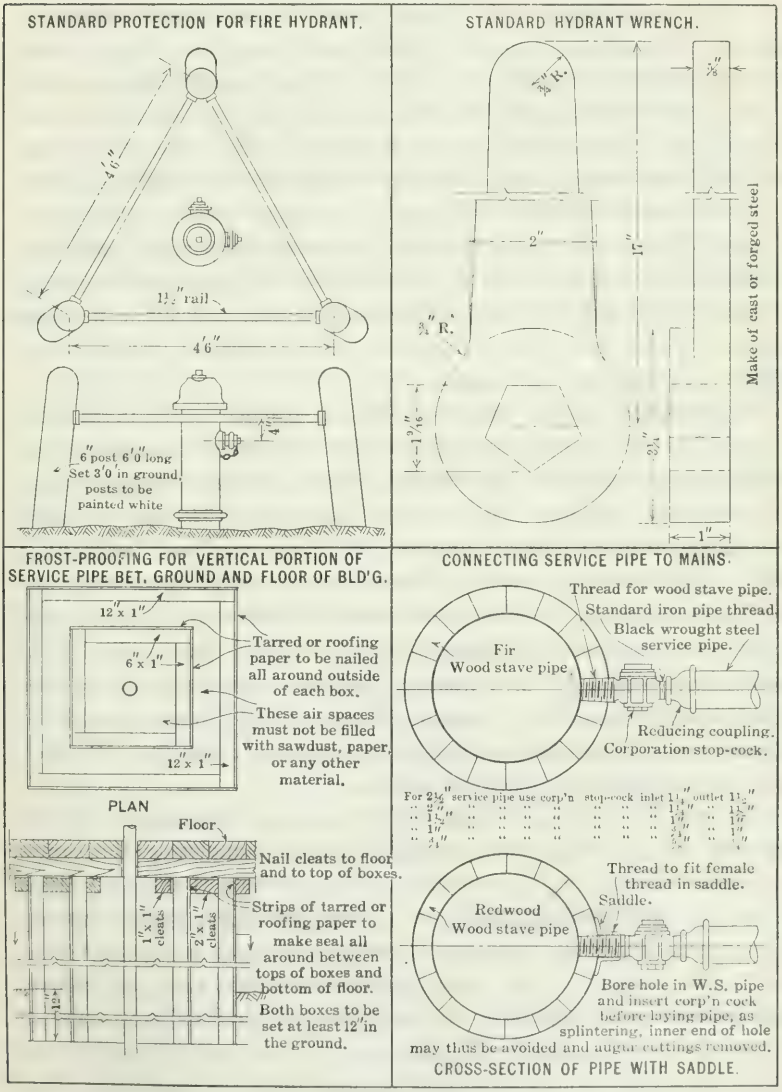

STANDARD PROTECTION FOR FIRE HYDRANT.
4′6″
1½″ rail
4′6″
6″ post 6′0″ long
Set 3′0″ in ground,
posts to be
painted white
STANDARD HYDRANT WRENCH.
¾″ R.
⅞″
2″
17″
¾″ R.
1 3/16″
2¼″
1″
Make of cast or forged steel
FROST-PROOFING FOR VERTICAL PORTION OF SERVICE PIPE BET. GROUND AND FLOOR OF BLD'G.
12″ x 1″
6″ x 1″
12″ x 1″
Tarred or roofing paper to be nailed all around outside of each box.
These air spaces must not be filled with sawdust, paper, or any other material.
PLAN
Floor
Nail cleats to floor and to top of boxes.
1″ x 1″ cleats
2″ x 1″ cleats
Strips of tarred or roofing paper to make seal all around between tops of boxes and bottom of floor.
12″
Both boxes to be set at least 12″ in the ground.
CONNECTING SERVICE PIPE TO MAINS.
Thread for wood stave pipe.
Standard iron pipe thread.
Black wrought steel service pipe.
Fir
Wood stave pipe
Reducing coupling.
Corporation stop-cock.
For 2½″ service pipe use corp'n stop-cock inlet 1½″ outlet 1½″
" 2″ " " " " " " 1½″ " 1½″
" 1½″ " " " " " " 1″ " 1″
" 1″ " " " " " " ¾″ " 1″
" ¾″ " " " " " " ½″ " ¾″
Thread to fit female thread in saddle.
Saddle.
Redwood
Wood stave pipe
Bore hole in W.S. pipe and insert corp'n cock before laying pipe, as splintering, inner end of hole may thus be avoided and augur cuttings removed.
CROSS-SECTION OF PIPE WITH SADDLE.

FIG. 9.

an elevation that the pressure available was sufficient for the domestic needs, and booster pumps were provided, where necessary, to raise the pressure and increase the volume available during the times of peak load and during fires. In order to conserve steel, redwood tanks of 100 000 and 200 000 gal. capacity were in most cases utilized to furnish the necessary storage. These tanks could be shipped from California within a few hours of receipt of order, and could be quickly erected. They cost much less per gallon of capacity than either steel or concrete construction. All distribution mains were looped, the only dead ends being on the 6-in. pipe extending down into the corrals to furnish water to the water troughs. A hydrant was set on the end of each of these 6-in. pipes, but the only structures in the neighborhood were the skeleton sheds or shelters for the animals, and the value of these sheds did not justify any considerable outlay for fire protection. These 6-in. pipes, therefore, were essentially service pipes rather than mains.

In planning the fire protection it was deemed wise, in view of the size, character, and distribution of the buildings, to provide plenty of hydrants, so that there might be a large number of small streams with pressures sufficient to give good horizontal projection or reach, and placed so as to make it unnecessary to use any hose longer than 300 ft. The hydrant branches themselves were only 18 in. long, and the friction saved by reducing their length justified the use of 4-in. pipe for the branch, feeding a hydrant with two $2\frac{1}{2}$-in. hose outlets. The nozzle tips were $\frac{7}{8}$-in. in diameter, and the combination thus used accomplished the desired purpose of providing a large number of streams quickly available through short lengths of hose, and with first-class pressure right up to the nozzle tip.

The use of wood-stave pipe, which was made compulsory by the necessity of conserving steel and iron and of completing the camps by September 1st, 1917, resulted in a large saving of money. Much of the pine pipe manufactured east of the Mississippi proved unsatisfactory, and, generally speaking, it was not as good as the pipe from the Pacific Coast. Of the latter, experience would seem to indicate that redwood was superior to fir, and that the air-dried was superior to the kiln-dried redwood. Having in mind the fact that experience proved that wood pipe of good quality, properly laid, showed very little leakage, and that the life of such pipe, thus laid, may safely be taken

at about 20 years, it would have been wise, in the writer's opinion, to use this pipe for the camps, simply on account of the saving in cost and time of delivery, even had there been no restriction on the use of cast-iron and steel pipe.

Sanitation.

Although the importance of having the water supply completed by September 1st, 1917, demanded the utmost speed, the question of the sanitary quality of the supply was never overlooked. This office constantly kept in touch with the Sanitary Corps of the Medical Corps of the Army, and there was at all times the fullest possible co-operation with the officers of that corps. Whenever there was the slightest suspicion of danger of pollution, the office that discovered it first immediately informed the other, and both then united in a determined effort to remove the danger.

Great credit is due to the supervising engineers, as well as to the engineers for the contractors, and to the sanitary officers assigned to the camps, for the efficient manner in which they safeguarded the purity of the temporary water supplies used during the construction period.

In addition to the care exercised by this office in the selection of the source of supply, and in purifying the water wherever necessary, particular stress was laid on the following sanitary precautions:

(*a*) The covering of all storage reservoirs in order to prevent accidental or intentional pollution and the formation of algæ;

(*b*) The furnishing of chlorinators to all cantonments or camps, even when the supply might be regarded as normally safe;

(*c*) The prohibition of the use of wells in shallow strata on inhabited water-sheds, or in any other stratum or location in which it might be possible for the wells to become contaminated;

(*d*) The absolute prohibition of any distribution mains carrying polluted water, whether for fire protection, or industrial, or any other use; and of any outlet or fixture from which such water might be drawn; or of any arrangement that would permit the later introduction of any such outlet or fixture. This ruling was often difficult to enforce, but it was always insisted upon.

The results accomplished by the careful consideration thus given to water-works sanitation have been discussed in the technical press.*

The inoculation of the troops for typhoid and para-typhoid by the Medical Corps greatly reduced, but did not entirely remove, the danger of their contracting these forms of water-borne diseases. There was no inoculation against other water-borne diseases, and civilians were not usually inoculated at all.

Water Systems for National Guard Camps.

The work on the water systems of the sixteen National Army cantonments had only fairly commenced when it was decided, late in June, 1917, to construct sixteen tent camps for the National Guard troops. The Construction Division was ordered to have five of these camps ready to take troops by August 1st, five more by August 15th, and the remaining six by September 1st. The difficulties of the problems incident to this new construction were enormously increased by the fact that the orders for water-works materials required by the cantonments had so exhausted the available stock and overtaxed the capacity of the various manufacturing plants that it was impossible for them to furnish any considerable quantity of similar materials within the time allotted for the construction of the National Guard camps. Therefore, it became necessary to use, in so far as possible, materials which had not hitherto been drawn upon. After a conference with prominent representatives of the manufacturers of wrought-steel pipe, it was decided to use 6-in. wrought-steel pipe for the distribution mains for eleven of these tent camps, to use 6-in. cast-iron pipe for the two camps which were in the Birmingham district, and to use 6-in. redwood stave pipe for the two camps in California and the one at Deming, N. Mex. The steel companies rolled and shipped 500 000 ft. of 6-in. steel pipe in a little more than 6 working days. The cast-iron pipe manufacturers shipped about 80 000 ft. of 6-in. cast-iron pipe within a not much greater period, and the wood-stave pipe manufacturers made correspondingly good deliveries of their product; so that a considerable part of the necessary material was on the way to the camps, in some cases before any constructing quartermaster, supervising engineer, or contractor, was on the ground, and in practically all cases before any plan showing the topography of

* Editorial in *Engineering News-Record*, April 18th, 1918, entitled "Our Military Camps Show Remarkable Typhoid Fever Record."

the camp site was available. In this matter, as in all others in which the Materials Branch of the Construction Division had a part, the efficiency of that branch was splendidly demonstrated.

Enlargement of Construction Division Activities.

As the National Army and National Guard camps neared completion, work was begun on Camp Hill and Camp Stuart, near Newport News, Va.; Camp Merritt, at Tenafly, N. J., and Camp Johnston, near Jacksonville, Fla.; the first three being embarkation camps and the last a school of instruction for quartermasters. Camp Meigs, in Washington, was also built at about this time, and then began the work of building warehouses, terminals, munition depots, arsenals, plants for the manufacture of gas, bag-loading plants, proving grounds, and all the other projects entrusted to the Construction Division, the field of activities of which was enlarged to include all construction work for the Army in the United States and its insular possessions.

The construction of the embarkation camps marked a change in the size and grouping of the buildings in each unit, and the typical layout of water mains was changed to suit the new building plans, the principal change being the use of three lines of mains, instead of two, through each unit. This resulted in still better fire protection, and in a great reduction in length of service pipes.

Some Typical Working Plans.

The personnel in the office having been enlarged at about this time, it became possible to prepare drawings in much greater detail, and the office was thus enabled to make plans showing much more clearly than formerly the arrangement of pumps, motors, and piping, in or near the various pumping stations or reservoirs.

A drawing of this sort could be turned out in a very few working hours by the specially trained men in this office, and these drawings invariably saved a great amount of labor in the field. Furthermore, these layout plans, which were sent to each job with the manufacturers' detailed foundation plans for the motors, internal combustion engines, and pumps, prevented serious mistakes on the part of the construction engineers, who, although unquestionably competent in many other lines of work, were naturally often more or less unfamiliar with the special field of water-works design, and were so fully occupied with

their construction work that they could not give the time for the development of carefully studied plans.

The plans shown on Fig. 10 are for a filter plant and pumping station of very small capacity, and are reproduced here more because

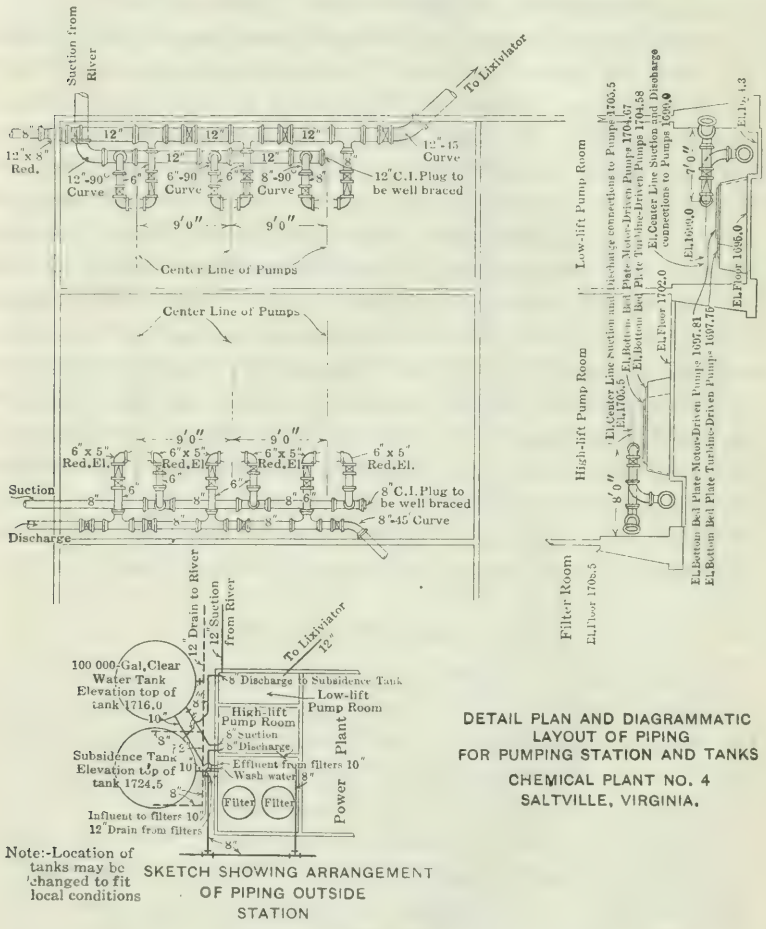
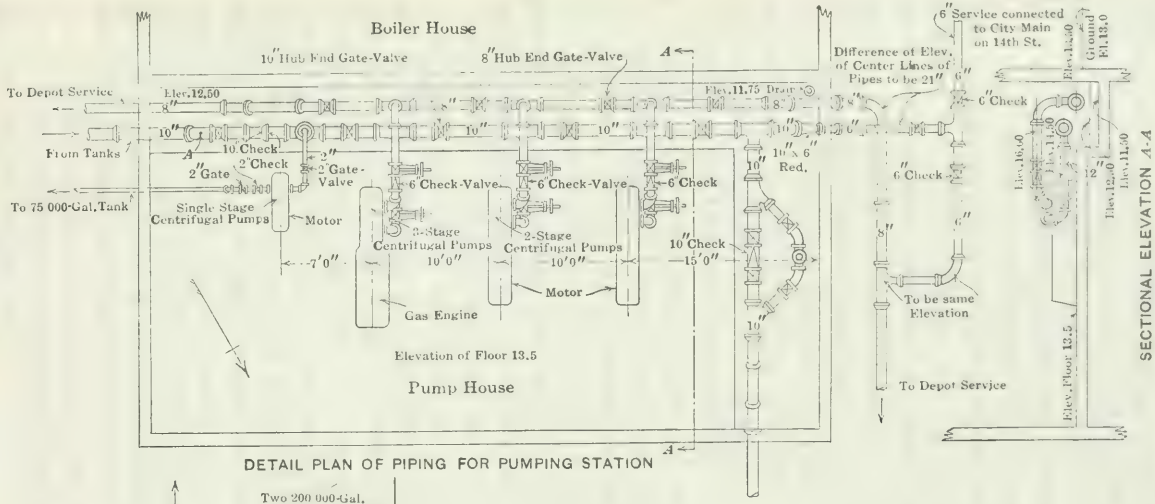
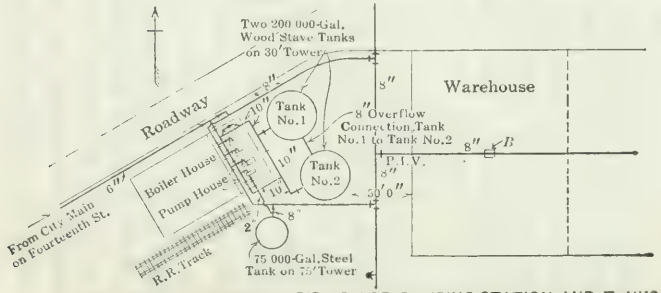

Suction from River
To Lixiviator
12" x 8" Red.
12"-90° Curve
6"-90° Curve
8"-90° Curve
12"-45° Curve
12" C.I. Plug to be well braced
9'0"
Center Line of Pumps
6" x 5" Red. El.
Suction
Discharge
8" C.I. Plug to be well braced
8"-45° Curve
Low-lift Pump Room
High-lift Pump Room
Filter Room
El. Floor 1706.5
El. Center Line Suction and Discharge connections to Pumps 1706.5
El. Bottom Bell Plate Motor-Driven Pumps 1704.67
El. Bottom Bell Plate Turbine-Driven Pumps 1704.58
El. Center Line Suction and Discharge connections to Pumps 1696.0
El. Floor 1696.0
El. Pipe 1695.0
El. Floor 1706.0
El. Center Line Suction and Discharge connections to Pumps 1706.5
El. Bottom Bell Plate Motor-Driven Pumps 1697.61
El. Bottom Bell Plate Turbine-Driven Pumps 1697.76
8'0"
7'0"
12" Drain to River
12" Suction from River
To Lixiviator 12"
100 000-Gal. Clear Water Tank Elevation top of tank 1716.0
Subsidence Tank Elevation top of tank 1724.5
5" Discharge to Subsidence Tank
Low-lift Pump Room
High-lift Pump Room
8" Suction
8" Discharge
Effluent from filters 10"
Wash water 8"
Influent to filters 10"
12" Drain from filters
Filter
Power Plant
Note:-Location of tanks may be changed to fit local conditions
SKETCH SHOWING ARRANGEMENT OF PIPING OUTSIDE STATION
DETAIL PLAN AND DIAGRAMMATIC LAYOUT OF PIPING FOR PUMPING STATION AND TANKS
CHEMICAL PLANT NO. 4
SALTVILLE, VIRGINIA.

FIG. 10.

the plant was an unusually complete one than for any other reason. It will be noted that there was a small sedimentation tank, two filter tanks, a clear-water tank, and three low-lift and three high-lift pumping units. It was the practice to eliminate from plans of this sort all unnecessary lines, or dimensions, or lettering, but to indicate what-



DETAIL PLAN OF PIPING FOR PUMPING STATION



DIAGRAMMATIC LAYOUT OF PIPING FOR PUMPING STATION AND TANKS

FIG. 11.

DETAIL PLAN AND DIAGRAMMATIC LAYOUT OF PIPING FOR PUMPING STATION AND TANKS. BALTIMORE DEPOT WAREHOUSES.

Note:-All gate valves within pump house to be outside Screw and Yoke. Pilot tube from altitude valve to be connected to 10" suction main at *A*.

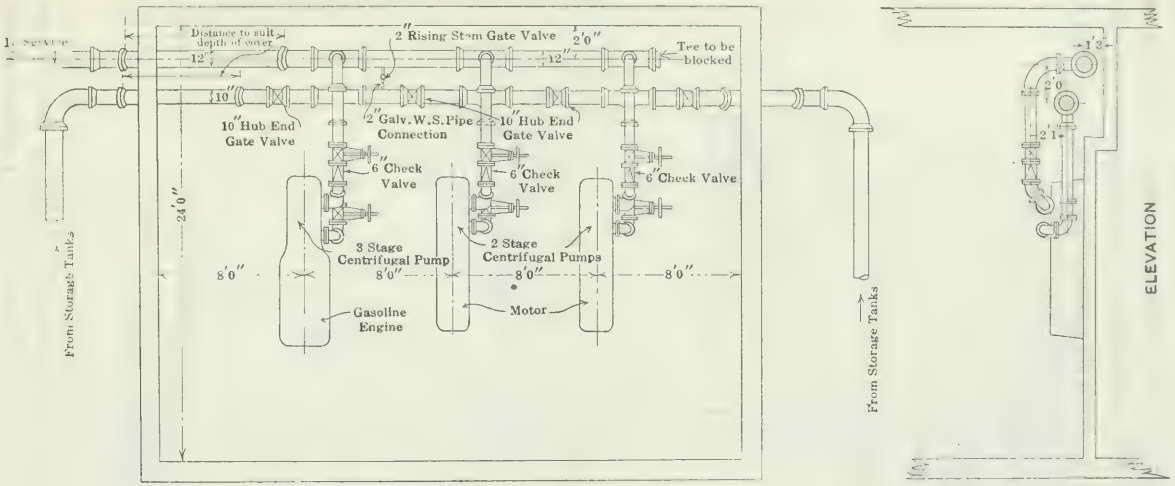
ever was shown so clearly that the drawing could be easily understood by the men in the field.

Fig. 11 shows plans for the pumping station, storage tanks, motor-driven pumps, gasoline-engine-driven pump, and piping and connections. In this case the supply came from a city main, the water from which flowed into the first of two wooden tanks, each having a capacity of 200 000 gal. The pipe connections were arranged so that the first tank was always kept full, the second tank being filled only by the overflow from the first one. In this way there was always at least 200 000 gal. of storage for fire purposes, in addition to whatever storage might happen to be in the second tank, which took care of the peak loads due to domestic use, and, in addition, also to such supply as the city mains might be capable of delivering to the suction of the fire pumps. The small pumping unit was utilized to keep the small sprinkler tank full at all times.

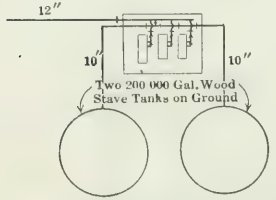
The pumping station and tanks shown in Fig. 12 were for fire service only. The supply in this case was from wells, and the ordinary domestic pressures carried were more than sufficient to fill the two 200 000-gal. tanks adjacent to the pumping station, so that there were always at least 400 000 gal. in reserve for extinguishing fires. There were two electrically-driven and one gasoline-engine-driven centrifugal fire pumps, taking their supply from the tank.

Fig. 13 shows plans for pumping station and tanks, in which it will be noted that there are two 200 000-gal. storage tanks near the surface, and a 75 000-gal. elevated steel tank. The supply was taken from the city mains, the pressure in which was sufficient to fill the low storage tanks, but not always sufficient to put water in the elevated tank; and a booster pump was provided to fill the latter and give the necessary domestic pressure. In case of fire the two fire pumps could take their supply from the two 200 000-gal. tanks, supplemented by such water as might come in from the city's mains.

It will be noted that practically all the regular service pumps were motor-driven centrifugals. These were operated, as a rule, by purchased current. Steam machinery and electric generators could not be had, whereas electric motors and centrifugal pumps were obtainable on reasonably short notice, and the combination made the cheapest of all pumping units. At least one internal combustion engine was usually provided as a reserve against a break-down in the electric service.



PLAN



DIAGRAMMATIC LAYOUT OF PIPING FOR PUMPING STATION AND TANKS

Note:-All gate valves inside of Pumping station to be outside Screw and Yoke

DETAIL PLAN AND DIAGRAMMATIC LAYOUT OF PIPING FOR PUMPING STATION No. 2 AND TANKS CURTIS BAY ORDNANCE DEPOT CURTIS BAY, MD.

FIG. 12.

Bell and spigot pipe and specials were used wherever possible, because material of this class gave greater flexibility for making connections, cost less, and could be more quickly obtained, than any other.

Plate X is a plan of a 2 000 000-gal. filter plant and pumping

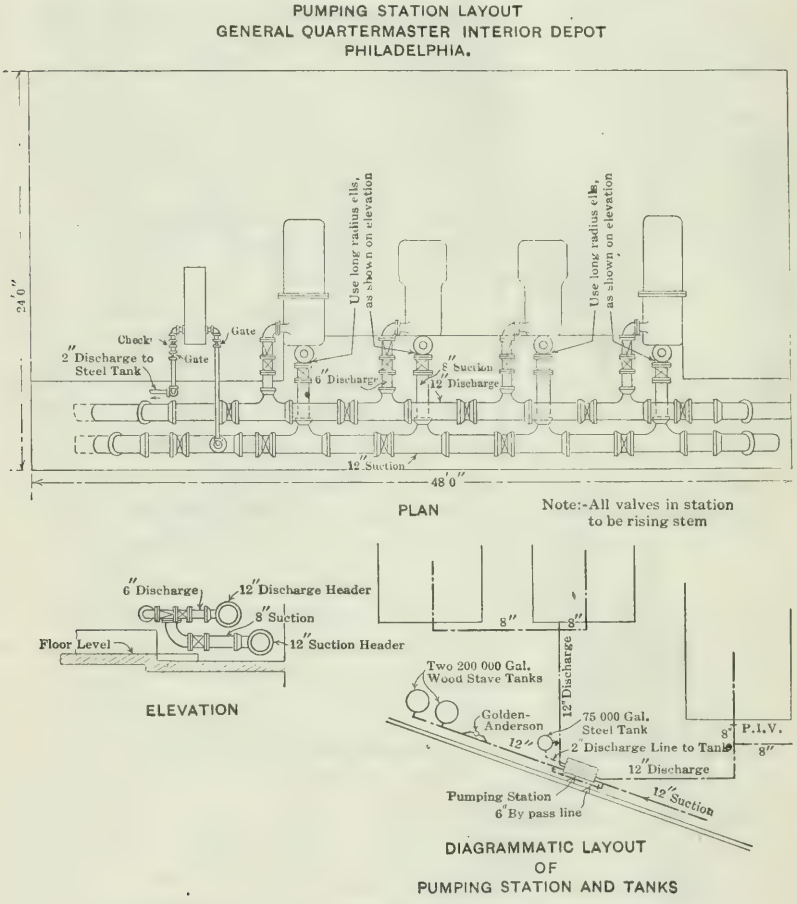
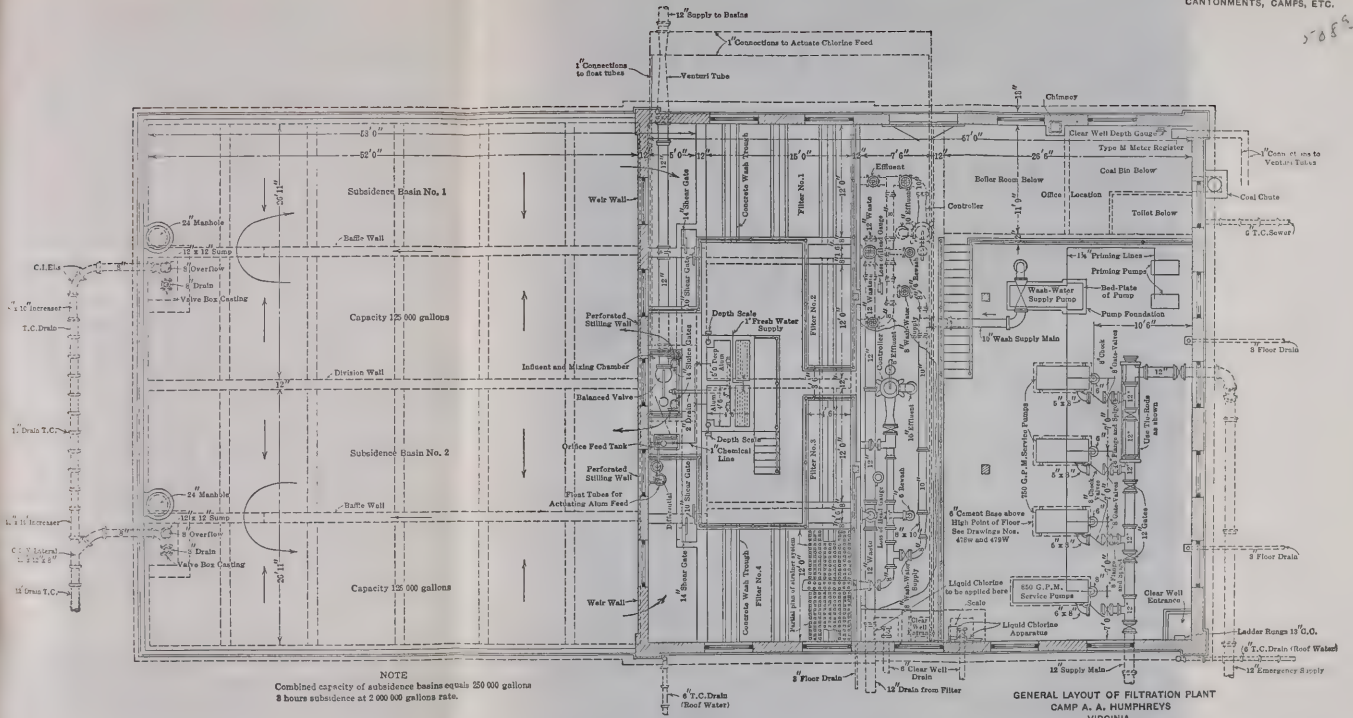


FIG. 13.

station. This plan was prepared by the contractors for the filter equipment, in accordance with instructions from the writer, and embodied two somewhat unusual features, both of which had been previously incorporated in plants designed by the writer before the war, and while he was still in private practice.



NOTE
Combined capacity of subsidence basins equals 250 000 gallons
8 hours subsidence at 2 000 000 gallons rate.

GENERAL LAYOUT OF FILTRATION PLANT
CAMP A. A. HUMPHREYS
VIRGINIA.

The first of these features was the arrangement of the mixing chambers and sedimentation basins. It will be noted that there were two sedimentation basins, each of which had a baffle-wall extending from the pumping station building toward the far end of the sedimentation basin. The two mixing chambers, one for each basin, were just inside of the rear wall of the building and near its center. The coagulant was added to the water in these mixing chambers, and the water in each basin then flowed away from the building around the end of the baffle-wall and back again to an effluent chamber similar to the mixing chamber, and likewise enclosed within the walls of the building. Large windows in the rear wall of the building provide abundant light over the mixing and effluent chambers, and it is thus possible for the filter operator, without leaving the building, to study, side by side, the influent and effluent water, and thus obtain an excellent idea of the way in which the floc is forming. This is a feature which has always appealed strongly to every filter operator.

Another somewhat unusual feature of the design of this plant is the open pipe gallery, which has the same advantages over the usual dark and concealed pipe gallery that open has over concealed plumbing.

In addition to the wash-water pumps, there are three electrically-driven and one gasoline-driven centrifugal pumps, all in the same main room as the mixing and effluent chambers, filter tanks, operating platform, and pipe gallery; and the whole plant is thus about as compact and accessible as it would be possible to make it.

The illustrations accompanying this paper show less than 1% of all the water-works plans prepared in the Washington office. They were selected for reproduction because they gave a fair idea of the general character of the work done. It should be noted, however, that none of the larger projects is shown, the plans for these being so large that they could not well be reduced to a scale suitable for reproduction in this paper.

Consumption of Water.

The determination of the quantity of water to be supplied to the National Army camps has already been mentioned. The specifications provided that the water-works systems should be capable of furnishing at all times at least 55 gal. per capita per day. This included an allowance of 15 gal. apiece for the horses and mules. As there was

about one animal for every three men in a divisional unit, the net allowance per man was 50 gal. per day.

This is about 80% more water than was supplied in the British, French, Belgian, Canadian, or German cantonments, and is larger than the quantity recommended by Capt. Francis C. Harrington.*

It is a well-recognized fact that Americans use, and waste, more water than any other nation; but there are many cities in the United States, where meters are in fairly general use, and in which the average daily per capita consumption is less than 50 gal., including all water wasted, leakage in mains and service pipes, water for flushing sewers, sprinkling lawns and streets, public and private fountains, the water used by the Fire Department, and all sorts of industrial, commercial, and manufacturing uses, these latter being often quite large in quantity; in fact, every drop of water that enters the distribution mains at the pumping station.

The experience of the Construction Division with the Army camps proves conclusively that the allowance of 55 gal. per capita per day to such camps was a very liberal one. A little study will show that it would be almost impossible for any man to use legitimately more than 20 to 25 gal. per day, and that all water used in excess of this quantity must represent leakage and waste. It is impossible to stop all leakage and waste, but it ought to be, and is, easy in such camps to keep the total consumption down to twice the maximum legitimate quantity actually required by the men.

When the sixteen National Army camps were first occupied there were only a few of them in which the consumption of water exceeded 55 gal. per capita per day, and in several cases the daily per capita consumption was less than 40 gal. At this time there was practically no check on the use or waste of water, as the utilities organization had not yet been perfected. Furthermore, it was not then possible to secure effective utility management, by reason of the fact that the utilities officers had to report to the Camp Quartermasters, which meant that they were under the Commanding Officer at the camp. As the chief interest of these Commanding Officers was to train troops for France, they were quite naturally not always willing to furnish the utilities officers with a sufficient personnel of non-commissioned officers

* In his paper on this subject, published in *Professional Memoirs*, Corps of Engineers, Vol. 8, No. 39.

and enlisted men to maintain the utilities in proper operating condition and to check leakage and waste. During the severe winter of 1917-18, the daily per capita consumption increased greatly in most camps, and in some instances exceeded 100 gal. This was due partly to leaks in plumbing, but largely to the fact that water was allowed to run to waste to prevent freezing. In the four most northerly camps, which had central heating systems, the water used for heating was not returned but was blown to waste, and the water wasted in this manner alone amounted to 600 000 gal. per day per camp during cold weather. By the end of the winter the utilities organizations in each camp had begun to attain their stride and make their influence felt, their efficiency being increased enormously by the fact that they were now reporting to the Maintenance and Repair Division of the Construction Division, instead of to the Commanding Officers at the camps.

In the early summer of 1918, orders were sent to the Commanding Officers at every Government activity, instructing them to co-operate in every way possible with a view to checking leakage and waste of water. The effect of these orders was soon apparent, and at practically all the camps the daily per capita consumption was reduced to less than 55 gal. without the slightest inconvenience or shortage of water for any one. As time permitted, the utilities officers, working under the direction of the Maintenance and Repair Division of the Construction Division, conducted water waste surveys and made frequent inspections and repairs of plumbing. This work not only greatly reduced the waste of water, but showed conclusively that the excessive consumption was due almost entirely to waste and to leaky fixtures, and that the underground leakage was almost invariably negligible.

Changes in National Guard Camps.

When the National Guard camps were originally laid out there were no barracks, the men being quartered in tents; and there were no sewerage systems. This meant that there was no necessity for providing either the same standards for fire protection, or the same quantities of water, as were called for by the National Army camps, an allowance of 30 gal. per capita being ample for them. Nevertheless, in laying out the distribution systems for these camps, the possibility that they might ultimately have sewerage systems, and that they might have to be converted from tent camps to barrack camps, was not over-

looked, and the two lines of 6-in. distribution mains running through each unit in the camp were laid along the outer edges of the units and in such manner that, later, should the changes just referred to be made, the water supply systems could be quickly brought up to the standards then required by laying an additional main of larger size through the center of each unit.

Early in 1918, the officers in charge of the Construction Division foresaw the probability of improvements in the National Guard camps. No orders for such improvements had been issued, however, and no funds were available. The one thing that seemed certain was that, when the orders did come, the changes would have to be made in a hurry. With a view to being prepared to reinforce the distribution systems in the Guard camps with the least possible delay whenever the order should come, this office endeavored to arrange some plan by which the necessary redwood pipe might be manufactured and shipped from California to some advantageously situated eastern point from which immediate deliveries could be made to the various National Guard camps. Unfortunately, however, there was no authority for any such action.

After nearly two months had passed in fruitless efforts to perfect the arrangement just referred to, that plan was abandoned, and it was decided to place an order for 300 000 ft. of redwood stave pipe of assorted sizes for the account of a large Quartermaster terminal project which had been recently authorized and the funds for which had not yet been drawn upon to any great extent. It was the intention to have the pipe actually made and shipped to this Quartermaster terminal, and thereafter to divert it, either in transit, or by shipment from the Terminal, to the various camps, to the account of which it could then be charged.

In the meantime plans were prepared in this office for the reinforcing of the distribution system and for such other changes in the water supply systems at the camps as might be rendered necessary by their conversion from tent camps to barrack camps and by the introduction of sewerage systems.

In August, 1918, this office was ordered to prepare immediately estimates of the cost of the changes in the water supply system of twelve National Guard camps, incident to the changes previously referred to; and with the plans for these changes already prepared, the office

was able to deliver the estimate required within two hours. Practically none of the pipe ordered for the account of the Quartermaster terminal, but actually required at the National Guard camps, ever reached the terminal. At the time of the shipment of this pipe the freight congestion on the railways was greater than ever before in the history of the country, so that it took from 6 weeks to 2 months to deliver pipe from the Pacific Coast to points east of the Mississippi River. A very serious delay was avoided in the manner just described.

Contracts for Water Service and for Extensions.

In a great many instances the water supply for the Government activities was taken from existing water-works systems which were owned either municipally or privately. In all such cases, special contracts had to be made for water service, and whenever the quantity of water desired was large, it was almost invariably necessary to embody in the contract for water service provisions for extensions to the existing plant, in order to enable it to meet the increased demands.

It was usually impossible for the owners of the existing system to finance the desired improvements, and these contracts, therefore, provided that the Government would make the required extensions and turn them over to the Water Department for operation as soon as they were completed. The contracts also provided that, within a certain reasonable time after the termination of the war with Germany, the owners of the existing plant should purchase the extensions from the Government on terms which fully recognized the fact that the extensions were made as a result of the emergency occasioned by the war. The interests of the owners of the existing water systems were thus fully and fairly recognized.

As a general rule, the rates paid by the Government for water service were unusually low, especially in view of the enormously increased cost of furnishing the supply. In practically every instance the rate at which water was sold to the Government was the lowest on the local schedule, and in many cases the Government rate was made lower than the lowest schedule rate, a condition which was usually justified by the fact that the Government was the largest local consumer. So far as the writer's recollection serves him, there was no large Government

activity which paid in excess of 15 cents per 1 000 gal., this rate being paid in one instance for water which had to be pumped three times, filtered, and chlorinated. The lowest rate paid for water was 4¼ cents. In many instances the rate was less than 8 cents.

A number of contracts were drawn in which other departments, such as the Navy, the Shipping or the Housing Corporation, were jointly interested with the Army; and these contracts involved negotiations with the several departments, as well as with the owners of the existing water systems.

Summary.

The office of the Advisory Engineer for Water Supply, Construction Division, has handled for the various branches of the Government, water supply problems on more than 275 different projects, involving the construction, in round numbers, of nearly 1 100 miles of mains from 6 to 30 in. in diameter; 700 miles of service pipes from ¾ in. to 2½ in. in diameter; 650 000 000 gal. per day in pumping capacity; 67 000 000 gal. in capacity of concrete, steel, or wood reservoirs or tanks; 3 000 000 000 gal. capacity in impounding reservoirs and more than 40 000 000 gal. daily capacity of filters.

In an article prepared by the writer* late in 1917, the following statements were made with reference to the cost of the water-works systems for the sixteen National Army Cantonments:

"It may also be stated that the cost of reproduction new of the average water-works system in a city of 40 000 inhabitants, at present prices for materials and labor, and exclusive of service connections, would average over $2 000 000, or more than $50 per capita. As nearly as can be ascertained at present, the cost of the water-works of the 16 National Army cantonments, including service connections to within 5 ft. of the buildings, has been less than $11 per capita, or say about one-fifth as great as the reproduction cost new to-day of the average municipal water-works system.

"Perhaps 25% of this great difference in cost of the entire system is due to the fact that about 70% of the distribution mains have been wood-stave pipe instead of cast-iron pipe, the cost of wood-stave pipe being on the average about 60% of that of cast-iron pipe. The greater density of population in the camps has also contributed materially to reduce the cost of their water-works. But when every possible allow-

* *Engineering News-Record*, January 3d, 1918, p. 35.

ance has been made, the comparison of cost cannot be other than favorable to the camps.

"Finally, and infinitely more important, is the time comparison. The time allowed for planning and completing a water-works system for a city of 40 000 population would, under ordinary circumstances, be about three years; whereas with the most difficult material, labor and transportation conditions ever known in this country, the water-works for the 16 Army cantonments have been planned and completed in a little over three months."

From the nature of the case it has been impossible to secure any data, even approximately accurate, of the cost of later water-works systems built by the Construction Division. There can be no doubt of the fact, however, that the unprecedentedly bad weather conditions in the winter of 1917-18, the epidemic of influenza, and, above all, the constantly increasing cost of labor, coupled with its steadily diminishing efficiency, have all combined to increase the cost of work, not only on the projects carried out by the Construction Division, but on every project of whatever nature that has been undertaken in the United States.

In round numbers, about 4 000 000 troops and 2 000 000 civilians have been supplied with water by plants designed and built by the Construction Division. Up to date, this office has not been informed of any case of typhoid fever or other water-borne disease which has been attributed to any of these water supplies.

The barracks, mess halls and lavatories built by the Construction Division were necessarily of frame construction, and, therefore, very inflammable. Many thousands of fires have started, but in only three cases has a fire escaped beyond the building of origin. In each case the fire department at the camp succeeded in saving a part of the barrack building in which the fire originated and a part of the mess hall to which it spread. These fires were extinguished by the outside fire protection afforded by direct pressure from the water-works systems in the cantonment.

The records for 1918 show that the per capita loss by fire throughout the United States was $2.70, whereas the total per capita loss by fire at the Construction Division activities for the year ending June 30th, 1918, was only 67 cents.

The writer trusts that he has already made it clear that the work described in this paper was in no sense a one-man job. It would seem evident that results such as have just been set forth could not have been achieved without the loyal and intelligent co-operation of every one of the many men who bore a responsible part in the work, not only in the office, but in the field.

DISCUSSION

Mr. Fuller.

GEORGE W. FULLER,* M. AM. SOC. C. E.—Great credit is due to Col. Maury for the very efficient way in which he handled the water supply projects in the Army camps. It may seem, perhaps, at first sight, that the construction of water-works for these communities of some 40 000 people was a relatively simple matter, particularly when they are spoken of in a city like New York where such large water-works projects have been undertaken; but when one takes into account the fact that more than 30 camps, having an aggregate capacity of more than a million people, were supplied with water in a little more than 3 months, it is really a very remarkable record.

A part of the success of supplying water to these Army camps, carried out in the short time that it was, was due to the very level-headed consideration of these matters prevailing in the Washington office. That is to say, they did not try to do in Washington all that was to be done in the line of engineering. They got out only two or three pages of concise specifications as to fundamentals; they trusted the men in the field to adjust to local conditions; they were wise, in this respect, as well as in the local personnel selected and in the general oversight exercised to keep investments as small as practicable.

The speaker has seen a great many of the water-works enterprises of the Federal Government in the past two years, and knows that Col. Maury is entitled to much credit for the practical sense that he has exhibited at every turn.

Many of the engineers, at first, were inclined to try to introduce for the various branches of the military establishment schemes on the lines of metropolitan enterprises with which they were familiar, and which would provide more water than was actually needed; but Col. Maury was always at the helm, feeling his way along and heading off these large metropolitan water schemes that were not needed; and, at the same time, guarding against serious water shortage.

It is to be hoped that the methods which were established for handling these water supply projects will be recorded for the benefit of our military establishment for all time. Col. Maury also might have said quite a little about the operating steps taken to control the wastage, which was first sought to be effected by direct military order.

These details need not be recited here at great length; but, in addition to rigid inspection, many devices were introduced, such as putting marbles in the top of each header leading to showers and notching the end of the pipe on which the marble rests. The insertion of perforated metal blocks on top of headers was also used for reducing the quantity of water delivered to the large number of shower baths at the camps. There were about 15 000 fixtures in a single camp, and

* New York City.

Mr. Fuller. wastage in some of them might have interfered seriously with the continuity of the supply. One other interesting procedure for restricting waste was the very simple one of bending down the float in each flush tank in the lavatories, so that there was a smaller discharge from the tank. The main fact is that they got along comfortably with 50 gal. of water per capita daily, as originally specified. This is a very creditable record, and reflects the benefit of the adequate engineering direction of Col. Maury and his associates.

Mr. Norcross. P. H. Norcross,* M. Am. Soc. C. E.—The speaker, who has been working continuously for the past 22 months for the Construction Division of the Army, first as Supervising Engineer on Cantonments, and later as an Assistant to Col. Maury, had the opportunity, soon after the work began, of becoming intimately acquainted with what Col. Maury was doing, and the general specifications and instructions for supervising engineers, pertaining to water supplies at the Government enterprises; for these reasons he feels qualified to offer a few remarks.

At the beginning, few, if any, of the supervising engineers concurred in Col. Maury's suggestion, that an allowance of 55 gal. per capita daily was ample for all needs and demands, as they had gained their experience in American municipalities, and quite naturally felt that, not only was this quantity of water inadequate, but that such a suggestion was almost heretical. However, they accepted this figure, and constructed water supplies in accordance with the general specifications prepared by Col. Maury's office. The wisdom of the plan soon proved that 55 gal. per capita daily was sufficient for all the needs of the camps, and the early difference of opinion has been supplanted with almost unanimous approval by the consulting engineers and the Military Establishment.

Mr. Fuller has mentioned the importance of preventing the adoption of "metropolitan" ideas in the construction of camp utilities. Col. Maury realized this necessity at the beginning, and very quickly succeeded in impressing on the supervising engineers the fact that "time was the essence", and that costly refinements of design and construction would cause serious delays and were not essential to the building of cantonment water supplies.

One of the greatest economic municipal crimes of the United States is the extravagant use of water. In no city is this emphasized more acutely than in Washington, D. C., where the daily consumption runs as high as from 150 to 160 gal. per capita, and where practically 60% of the total water furnished is used by Government departments, for which no direct charge is made. The remaining 40%, which is used by the private consumer, bears the entire burden.

* Atlanta, Ga.

Mr. Norcross.

Much has been said regarding the speed with which the Construction Division supplied the War Department with camps. Speed was even more important in the construction of the water utilities, because of the vital necessity of establishing both temporary and permanent water supplies before the construction forces, and subsequently troops, could be quartered at the cantonments. To civilians, accustomed to the slow and systematic method in which cities and other bodies politic construct their utilities, the short time within which the water supplies were made available can only be accounted for as the result of a species of patriotic zeal and endeavor, unprecedented in history. This, in a large measure, was attributable to the cordial co-operation and enthusiasm that Col. Maury imparted to those who came in contact with him. The speaker and his fellow-workers could not help but be impressed with the necessity for urgent and more urgent action. To use a military expression, the *esprit de corps* of the Construction Division, composed practically of civilian engineers from all sections of the United States, is something that no one can explain, but it is a tangible memory that all can recall with pride.

Wherever it was possible, water was obtained from existing developed supplies, owned either by municipalities or private corporations. As a member of Col. Maury's staff, the speaker had the opportunity of negotiating, both with municipalities and public service companies, for the purchase of this commodity, and feels that some comments on the policy adopted by the Advisory Engineer for Water Supply in these negotiations may be of interest: Many of the National Army and National Guard camps were situated near municipalities, ranging in size from 20 000 to 60 000 people; consequently, if water was purchased from an existing supply, the demand on that plant was practically doubled. To the Advisory Engineer of Water Supply was delegated the powers of negotiating these contracts. To consummate these with fairness and equity to the seller, and, at the same time, to obtain reasonable rates and service for the Government, required, in addition to technical ability, unusual diplomacy and tact. The records of Col. Maury's office will show that there prevailed a closer harmony and co-operation on the part of the seller and buyer than has ever previously existed in public service dealings in America.

In many instances, due to increasing, or doubling, the personnel at the camps, the municipalities, or corporations, had to make a great increase in their normal facilities for supplying water. When these cases arose, the Government financed and constructed the work, and allowed the city, or private corporation, to reimburse on a basis of 75% of the actual cost to the Government, payments being made in water service, and interest charges on obligations thus incurred being deferred until after the cessation of hostilities.

Mr. Norcross. In conclusion, the speaker desires to say that Col. Maury has reflected great credit on the American Society of Civil Engineers, the Construction Division of the Army, and the Engineering Profession in general, by his able and fair-minded method of seasoning his technical work with the proper modicum of common sense and fair dealing.

Mr. Goldsmith. CLARENCE GOLDSMITH,* M. AM. SOC. C. E.—Because of the unfamiliarity of most of the Supervising Engineers with the laying of wood stave pipe, the work presented certain problems which had to be overcome with a minimum of delay. In those parts of the country where wood stave pipe had been used, it was customary to provide special bells, on valves, fittings, and adapters, which were tapered to receive the tenons of the particular pipe being laid. The diameters of the tenons of the several sizes of pipe of the several manufacturers were not uniform, and in most cases the individual manufacturers had a few patterns of the ordinary fittings with bells designed to fit the tenons of their pipe. In some cases the inner surface of the bell was only snagged and smoothed with a file; in a few cases the inside of the bell was machined. It is obvious that the machined bell made the better joint.

More than 1 100 miles of wood stave pipe were laid by the Construction Division, and, therefore, it was impossible to secure special castings and machine them in order to provide for the bends, branches, and adapters required.

Owing to lack of experience, together with the haste with which the work had to be done, and the resultant makeshift methods used in many instances, it was feared that the underground leakage would be excessive. This, however, did not prove to be the case, for as soon as the Maintenance and Repair Division got the operation of the water systems under control, tests showed that the average leakage in the 20 or more miles of mains in each of the several camps was not more than 200 000 gal. per 24 hours, and, in one camp, it was found to be about 100 000 gal. This rate does not compare unfavorably with that ordinarily found in distribution systems laid with cast-iron pipe.

In order to assure the tightness of the system, it is absolutely necessary to leave the trench open, with only a small quantity of filling replaced over the pipe between the collars, to prevent the line from floating, in case the trench becomes filled with water, and to keep the pipe line filled with water under little or no pressure for several days, in order to allow the staves to swell. When the line is ready to be tested, a pressure of about 50 ft. in excess of the working pressure should be applied, and all leaks which are found should be stopped by inserting plugs or wedges. It is desirable to have at least one man

* Chicago, Ill.

who is experienced in laying wood pipe, in order to direct the work and repair any leaks. Mr Goldsmith.

In some cases considerable objection was raised to keeping the trenches open long enough to permit the test, but the necessity of making it more than counterbalanced the inconvenience involved. If leaks are not stopped before back-filling, the escaping jet of water frequently forms a sand-whirl which quickly cuts the wire and causes the complete failure of a section of pipe. In one case a sand-whirl cut completely through a 1-in. corporation cock.

Service connections to fir pipe (which has a thick stave) were made by inserting a corporation cock with a wood thread into a hole of slightly smaller diameter which had been bored with a bit. The thickness of the shell of the redwood pipe is insufficient to assure the holding power in all cases, and it is desirable to have saddles, with outlets of the desired sizes, wound on the pipe to receive the corporation cocks. The standards developed for service pipes provide for screwing the iron pipe directly to the corporation cock. It was not considered necessary to provide lead goosenecks, and experience has shown that they were not necessary. The saving of material and labor thus effected was considerable.

The speaker is of the opinion that, even if it had not been necessary to conserve cast iron and steel, the use of machine-wound wood stave was justified, both because of its lower cost and because it could be laid so quickly. Although machine-wound wood stave pipe cannot be considered as desirable beneath permanent pavement, in closely built-up sections, it is certain that it is adapted for use in a much larger field than has previously existed, particularly for force mains and in undeveloped sections.

T. Kennard Thomson,* M. Am. Soc. C. E.—The Society is much indebted to Col. Maury for this very valuable paper. The work done by the Army and Navy engineers has wakened the whole profession, and cannot fail to have a broadening effect on future engineers as well as on those of the present time. Mr. Thomson.

The reference to placing the water and sewer pipes where they will always be exposed to view, makes the speaker hope that this practice will become much more general than it is at present. He remembers the time when it was impossible to reach the plumbing, in most cases, without doing great damage to the house, and, even now, it is impossible to reach water, gas, sewer, or other pipes without tearing up the street—an almost continuous operation. The sum paid for such work would be more than enough to pay for putting all such pipes in permanent tunnels, so that the streets would never have to be torn up, except to repair the roadway.

* New York City.

Mr. Thomson. A few years ago a water main broke, over a subway that was being built, and started to flood an adjoining cellar. It took three hours to get the city to turn off the water, and the only way a catastrophe was avoided was by constructing a dam of sand bags and diverting the water into the sewers.

Quite recently, when some caissons were being constructed on lower Broadway, New York City, the water level rose 9 ft. above normal, just as the compressed air was put on the work. At first, the Water Department thought that the work had caused the water to rise, which, of course, could not be. The work was continued, and the water level dropped back almost 11 ft. Some of the new subways are below the water level, and may or may not be affecting it, but it seems to be certain that the water level in lower New York City cannot rise and fall from 9 to 11 ft. under old buildings, which rest on the so-called New York quicksand, without sooner or later wrecking some of these buildings. It might be stated here that the ground-water in lower New York City used to stand always from 6 to 9 ft. above the high-tide level.

The object of these remarks is to lead up to the hope that, in the future, proper tunnels will be constructed for all pipes and wires, and that even the subways for cars will not be holes in the ground, but constructed so that there will be sidewalks and stores throughout their whole length, on both sides, and an opening, 2 or 3 ft. wide, in the center of the street, for the whole length of each block, so that the subways will always be attractive and well-ventilated, thereby greatly enhancing the value of the adjoining property and the health of the community.

Mr. Mason. W. P. Mason,* M. Am. Soc. C. E.—The speaker is particularly interested in Col. Maury's reference to the use of chlorine, as he is a great believer in some form of emergency defense, so that, in the event of sudden need, the people can be protected against a threatened catastrophe.

He is also greatly interested in what was said in reference to the daily quantity of water supplied, and is impressed by the wisdom shown by the author in deciding that 50 gal. per capita would be sufficient. In every community there are many who insist that water should be "as free as air", and many believe that large quantities are needed to flush the sewers. If they would only call to mind the little dribble that runs along the bottom of a large sewer, except in the case of a heavy rainfall, they would realize how small is the chance for sewer flushing under normal circumstances.

There is no reason for taxing a man for his neighbor's use or misuse of water, any more than there is for taxing him for his neigh-

* Troy, N. Y.

bor's consumption of gas; but, in the northern part of New York State, it seems to be absolutely impossible, so far as the speaker can find out, to keep the people satisfied with 100 gal. per capita per day. Perhaps they will feel differently after July 1st. The speaker does not know what the future will bring forth, in the matter of educating the public, but, to his mind, 100 gal. per day is enough for the average mortal. Mr. Mason.

J. Waldo Smith,* M. Am. Soc. C. E.—The speaker, unfortunately, has not been very close to any of this great work which Col. Maury has been doing; but from what he knows about it, it seems that the War Department was singularly fortunate in securing the few men gathered together in Washington, men of very broad experience and excellent judgment, who laid down the skeleton of the foundation for this work. Mr. Smith.

Secondly, the Department was fortunate in the personnel which was developed to carry out this work, and afterward to take charge of its maintenance and operation. This was done at great personal sacrifice on the part of a great many of these men, in many instances, as the speaker knows; and it is perfectly true, as both Col. Maury and Mr. Fuller have stated, that, following general directions, they were left largely to their own devices to develop those plants and expedite the work.

Col. Maury mentioned the form of contract under which this work was carried out. The speaker agrees with him that that was the only form of contract to be adopted under the circumstances; but he does not agree with the statement, in an address† by Gen. Marshall at the Annual Meeting, in January, in which he advocated the continuance of this form in general contract work after the war, because, in the speaker's opinion, the general adoption of such a form in ordinary work would be absolutely demoralizing to the whole contracting fraternity and very much against their interest, and, eventually, would lead to favoritism and discrimination, as well as the elimination of the gambling element, which is one of the principal interests in contracting.

Besides, there would be no chance for the man of great inventive resources, who is full of expedients and develops new methods of doing work.

To the speaker's mind it would result in a great deterioration of the contracting organizations. He does not know Col. Maury's opinion on this subject, but he was very much surprised to hear Gen. Marshall put it forth as a desirable feature. Certainly, the work of this Construction Division, of which so much has been heard during the last month or two, seems to appear as the bright and shining light of all the activities of the Army. This work seems to have been excellently planned and carried out, even to the details of operation and maintenance.

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* New York City.

† *Proceedings*, Am. Soc. C. E., February, 1919, pp. 164–171.

Mr. Smith. Of course, difficulties were encountered, but it seems that most of them were anticipated and prepared for beforehand; and when they were encountered, very prompt and effective methods were taken to remedy them.

Mr. Thurston. E. T. THURSTON,* M. AM. SOC. C. E.—The speaker's experience with the water problem in military camps was as a consumer only. At Camp Devens, Mass., where his unit was mobilized, during the bitter cold winter of 1917-18, water was abundant, and every desirable facility was afforded for its application to the needs of the men, thanks to the energy and foresight described in Col. Maury's paper. In France, however, the situation was quite different. Not only was there a scarcity of water, but the systems were primitive, following French practice. Strict economy was the watchword everywhere, although in less degree at the headquarters camps than elsewhere. In the wash-houses, water was delivered into overhead wooden troughs, about 8 by 6 in. in size, fitted with small wooden spigots at intervals of 2 ft. or so, which would emit a very small stream of water into the waiting hands of the washer. Wash-basins were not used, and usually guards were present to prevent the waste of water. Drinking water was chlorinated in canvas bags from which it was drawn as required; the drinking of unchlorinated water was prohibited.

Shower-bath houses, with water heaters, were provided, but in many cases were of little or no use on account of the water shortage. This was the case at all the camps at St. Nazaire, except the headquarters camp. On arriving with his company, the speaker was informed by the Adjutant that the bath-houses were not to be used on account of the shortage of water. Thus, thrown on our own devices, we overcame the difficulty and incidentally provided the necessary hot water for cleansing mess kits, by foraging some pipe and a barrel, constructing a water coil for the field stove, and carrying hot water in buckets to supply the wash-house tank. The men were withheld from their details in convenient detachments, so that each man had a warm bath once a week. This case serves as a fair illustration of the ingenuity and energy displayed by American troops in overcoming incidental obstacles that confronted them.

The record of achievement provided by Col. Maury's paper is noteworthy. There are many others in every branch of the Service lacking publicity, and, it is hoped that a comprehensive record of the achievements of reserve officers in connection with the war will be prepared by sympathetic and informed persons.

Mr. Boorman. K. M. BOORMAN,† ASSOC. M. AM. SOC. C. E.—The speaker, whose last duty overseas was with the Water Service of the Second Army,

* San Francisco, Cal.

† New York City.

Mr. Boorman.

feels that Capt. Thurston has given an inadequate idea of the water supply work in France. The Water Supply Service of the American Expeditionary Force was under the charge of Col. F. F. Longley, M. Am. Soc. C. E., who, the speaker believes, is known in New York City as a water supply engineer of some prominence. This service grew up with the development of the A. E. F.

During the first year, the water systems were usually built by the construction units locally engaged in building the camps and bases, as well as the general construction work for the camps. Of necessity, the supply varied with the size and importance of the camp, and those built entirely by the American army acquired better systems than those enlarged from old French camps using inadequate French systems as a basis. At the large aviation center at Romorantin, a field built on a new site by the Construction Division of the Air Service, water was supplied, a few months after the work was started, to all the buildings for drinking purposes and shower baths, through a modern system of piping from the Sauldre River.

On August 8th, 1918, the Water Supply Service was ordered to be organized, to be composed of "certain officers and certain engineer troops", to be a separate organization under the direction of the Chief Engineer of the A. E. F. By October, when both the First and Second Armies, fully organized, were operating on their own fronts, the Water Service was working thoroughly and efficiently. The 26th Engineers, with units of Pioneer Infantry attached to do the common labor, were assigned for this work in the Army areas.

"Water points" were established at all practical places. A typical procedure was to pump water from streams (such as the Rupt de Mad River in the Toul Sector) to a concrete settling basin, and run it through sand filters to a clear-water basin from which it was pumped into the pipe lines. It was usually necessary to chlorinate the water at the filters. From these basins the water was pumped to storage reservoirs accessible to the water tank trains of wagons and motor trucks, which were also part of the Water Service.

In many cases water was pumped through 4 and 2-in. pipe lines directly to the batteries firing on the German lines. During the last month of the war, the detachment of Co. B, 26th Engineers, near Thiaucourt, in front of Metz, connected their own pipes to miles of German pipe lines laid in the area gained in the St. Mihiel drive. This German piping was of sheet-iron and spiral-wound, usually about 3 in. in diameter; the American was the usual screw-fitting iron pipe. The pipes were buried to a depth of 1 m., for protection against frost and shrapnel.

The speaker hopes that, on the return of Col. Longley and Maj. A. H. Pratt, M. Am. Soc. C. E., Chief Water Supply Officer of the

Mr. Boorman. Second Army, they can be induced to present a paper on the work of the Water Service in France, of which the instances cited are only a small part.

Mr. Boucher. WILLIAM J. BOUCHER,* ASSOC. M. AM. SOC. C. E.—The Charleston Port Terminal, at Charleston, S. C., on the Cooper River, about 11 miles from the City Hall, is an exceedingly well planned rail and water terminal, consisting of an Animal Embarkation Depot, an Ordnance Depot, and a Quartermaster Depot. The property, Fig. 14, is traversed by the main line of the Seaboard Air Line Railway, is contiguous to the main line of the Atlantic Coast Line Railroad, and is also served (through connection) by the Southern Railway.

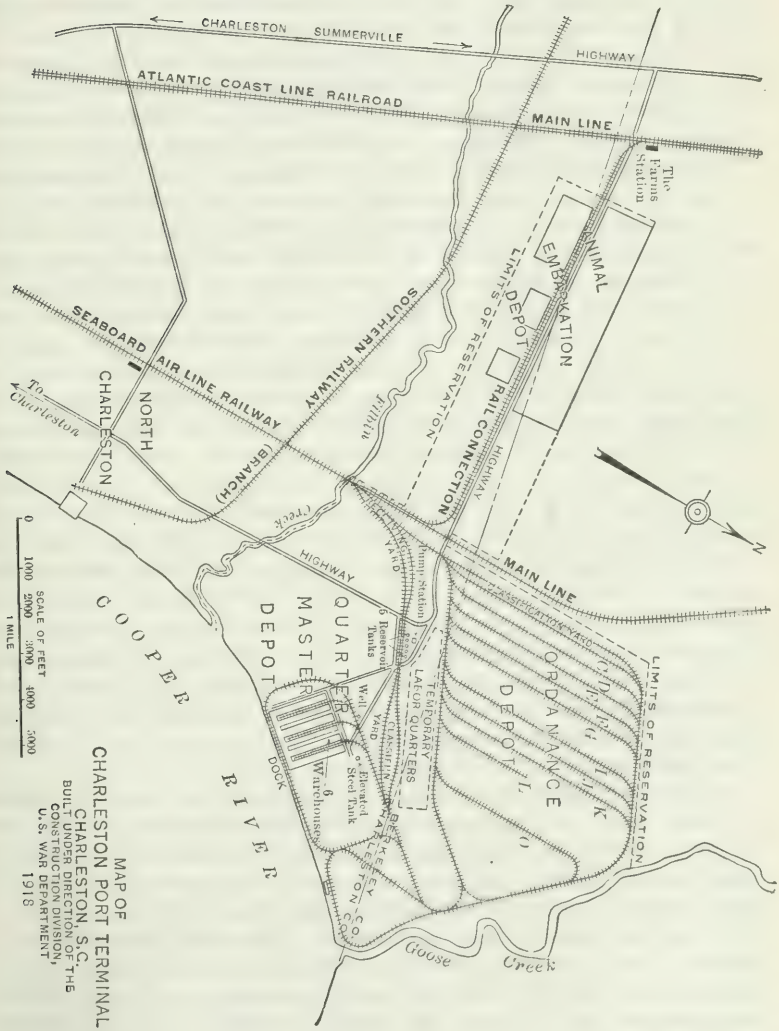
The Cooper River is a broad stream with considerable current, and, by a moderate amount of dredging, a channel depth of 30 ft. has been obtained. The ground is generally level, its elevation varying from 5 to 25 ft. above sea level; the surface is sandy, and the drainage is very good. When acquired, the land was partly covered with pine forest and partly devoted to agriculture.

The Animal Embarkation Depot or Remount (Mule) Station is at the westerly end of the reservation, and has accommodation for several thousand mules or horses. The pens or corrals are composed of simple fences, about 5 ft. high, without roofs. Hay is fed to the animals from a rack in the interior. The corrals are of varying sizes, according to the uses for which they are intended, and are joined by lanes with suitable gates. There are, in addition, the necessary office buildings, barracks for the troops caring for the animals, blacksmith shop, warehouses, feed buildings, and (open) hay sheds. Manure is shipped away in cars.

The Ordnance Depot covers an area of approximately 1 sq. mile, and is bounded on all sides by standard-gauge tracks, which, in turn, are connected by several parallel tracks. On each of the tracks, *D, E, F,* and *G,* there are eleven smokeless powder magazines, or forty-four in all. These are 32 by 96 ft., of wood-frame construction, covered with corrugated asbestos siding and a roof composed of gypsum blocks, covered with prepared roofing, rendering them fire-proof from the outside. The several buildings are separated from each other approximately 400 ft., in all directions. On tracks, *I, J, K,* and *L,* there are twenty-seven standard magazines, each 50 by 220 ft. These have brick pilasters supporting the wood roof-trusses, hollow-tile walls, and gypsum roofs (built in place, similar to reinforced concrete). On Track *O* there are eight primer and fuse houses, each being 32 by 96 ft. These have hollow-tile walls, wood rafters, and gypsum block roofs. There are seventy-nine buildings, of these three types, for the storage of explosives. There are, in addition, several guard-houses for the

* New York City.

Mr. Boucher.



MAP OF
CHARLESTON PORT TERMINAL
CHARLESTON, S.C.
BUILT UNDER DIRECTION OF THE
CONSTRUCTION DIVISION,
U.S. WAR DEPARTMENT
1918

FIG. 14.

Mr. Boucher.

use of troops patrolling the area, a garage and battery-charging station, and a commodious administration building.

For the use of the Quartermaster Department there were planned six warehouses, each 160 ft. wide and 1 200 ft. long, and two open sheds of the same size, but work on the latter was stopped at the time of the Armistice. The warehouses are of brick and hollow-tile wall construction, with wood posts supporting a wood roof, and are only one story in height. They are divided by fire-walls 100 ft. apart, making twelve rooms to a building. A complete sprinkler system is provided for these buildings. On each side of each warehouse there are railroad tracks and team or motor driveways. One of the buildings is two stories in height for about one-third of its length, and this second floor is used as the Administration Office for the officer in charge. The warehouses are parallel to each other and normal to the dock, which extends along the shore of the river; all the warehouses are joined by a long head-house, which, in turn, gives directly on the dock.

The dock, which is a marginal wharf, extends from the southerly side of the first warehouse, in a northeasterly direction (following the river bank) about 3 300 ft. It was originally planned to be of concrete piles with a reinforced concrete deck, but the plans were modified, and it has been constructed of creosoted piles with a timber deck. A standard gauge track, running the entire length of the dock, connects at both ends with the tracks on shore. For handling freight between warehouses or to ships, storage-battery trucks have been provided. Each carries about a ton, and, in turn, can haul several loaded trailers. Along the front of the dock, for its entire length, wood sheathing has been driven, forming a bulkhead, behind which has been deposited the material pumped up by the dredges in deepening and widening the channel. In this manner a large area of marsh land, overflowed at high tide, has been reclaimed, and the fill serves as a protection for certain untreated piles which were driven, the teredo being very active in the Cooper River.

The water supply for Charleston is obtained from the upper reaches of Goose Creek, and impounded by a dam approximately 20 miles from the city. Because of the greatly increased demands for water since early in 1917, and on account of the very dry summer of 1918, the supply proved entirely inadequate for the needs of the Terminal, and plans for increasing it were made early. Two wells were sunk in the Remount area and one near the Quartermaster warehouses; these are from 441 to 535 ft. deep, and supply a very good water, slightly impregnated with sulphur. In addition to these wells, an arrangement was entered into with the City of Charleston for increasing its supply, with Government aid. A dam has been constructed across the head-waters of the Ashley River and a 24-in. conduit of wood staves has been laid along the Charleston-Summerville Highway, connecting

with the supply pipes leading into the Terminal. The reservoir for the Terminal consists of five circular wood tanks, on concrete foundations, each having a capacity of 200 000 gal. These tanks are near the pumping station, and the water is distributed from the latter through cast-iron pipes varying from 16 to 6 in. in diameter, for drinking, sanitary, and fire purposes. Some 12-in. wood-stave pipe is used between the 24-in. main in the highway and the reservoir. There are eighty-nine hydrants distributed about the grounds. A 10-in. pipe runs along the entire length of the dock, the head-house has a 10-in. wrought-iron main, and the warehouses are protected by sprinklers, the supply for the latter being contained in a 200 000-gal. elevated steel tank near-by. Mr. Boucher.

During construction, temporary mains, up to and including 3 in. in diameter, were laid through the Ordnance area for mixing concrete, and 1 and 2-in. pipes supplied water to the inhabitants of the temporary camp. All these pipes have been removed. Permanent barracks for the use of stevedores (troops or civilians) have been erected, and in separate buildings there are lavatories and toilet facilities, similar to those at the several cantonments. There has also been constructed a power-house and refrigerating plant, and a commodious fire-engine house, containing a new model, chemical, motor-driven engine.

The Terminal was constructed jointly by The Degnon Contracting Company and The Mason and Hanger Contracting Company, on a cost-plus-fee basis, under the direction of the Construction Division of the War Department.

Morris Knowles,* M. Am. Soc. C. E.—The speaker has had the opportunity of reading a report by M. R. Scharff, Assoc. M. Am. Soc. C. E., on the water supply in the region of Brest, and is impressed with the accuracy of the statement relative to the materials and other facilities for water supply protection in France being far less adequate and complete than were those for similar work done in the United States. It must have been true that much of this work was carried on with insufficient quantities of absolutely essential materials. Mr. Knowles

The speaker cannot refrain from adding a word of commendation for the work of Col. Maury and his associates, or from expressing his gratification at having had the opportunity to serve in the Cantonment Construction Division water supply work under the direction of Col. Maury; and in commending this work, he wishes to speak especially of the independence and foresight with which Col. Maury and his staff assumed the responsibility of fixing the limit of use of water to be allowed in design. All know how difficult it is, in municipal water supply work, to convince people that a limited quantity of water constitutes a reasonable supply, and to combat the old fallacy that water

* Pittsburgh, Pa.

Mr. Knowles.

should be as free as air. There is no doubt that many felt, at the time of this decision, as the speaker did himself, that there was serious doubt whether a limitation of designs and installations to a basis of 50 or 55 gal. per capita per day could be successfully applied. It is clear that the success which justified this decision was due, in a large measure, to the restrictive measures inaugurated for preventing excessive use and for distributing the peak loads by planning hours of use for various purposes. It would be interesting if Col. Maury would enlarge on these restrictions in his paper, and on the difficulties encountered in enforcing them because of conflicting military orders by the commanding officers of troops. In the speaker's experience, considerable difficulty was encountered from this source, and, as a result, a shortage of water occurred during certain hours of the day for a large part of the time.

The method of supervising the work of which Mr. Fuller spoke was certainly admirable, in that it offered many helpful services while leaving to those at the different camps a large measure of discretion and responsibility. It was not always altogether easy, however, to confine this supervision within the limits of this conception. If time had been available to discuss every question, close supervision and detailed review would undoubtedly have resulted in more perfect and generally satisfactory designs. Under the circumstances, however, when everything had to be done in haste, it was inevitable that such close supervision and detailed review should have been, in some cases, impossible, and, in other cases, the cause, either of delay, or of resultant designs, suggesting the presence of two different and incompletely harmonized conceptions of the situation.

One of the most troublesome problems in connection with cantonment water supply was the provision of a temporary supply. In the beginning there was a tendency to commence work at different points, without consideration of the difficulty or impossibility of supplying water at these points from any reasonably near source. Similar difficulties were encountered with the construction camps, which grew very rapidly, and with the National Guard units which arrived on the ground early for the purpose of establishing guards and protecting property. Frequently, these latter units were commanded by officers of limited military experience who would wish to select camp sites which were not well adapted for the supply of water and the disposal of wastes. One of the things that kept the supervising engineer busiest at the beginning of the work was the location of such units so as to utilize in the best way the natural resources, and especially the shallow underground water resources, of the site. In spite of every effort, it was often impossible to avoid supplying such camps by transporting water in carts.

Mr. Knowles.

Referring to Mr. Smith's remarks about contracts, the speaker is in agreement with all those with whom he has been associated on war-time construction work in believing that the form of contract used was the only one that could have been successful under the circumstances. In the same way, it was necessary to have similar contracts for the housing developments which were constructed by various governmental agencies for the emergency. It does not follow, however, that such contracts are desirable for work of this character carried out under more nearly normal conditions—although the speaker believes that it is possible to draft contracts based on cost-plus-a-sliding-scale fee, which may be advantageous at all times. The Emergency Fleet Corporation, with which the speaker has recently been associated, is canceling many of its housing development contracts on the cost-plus basis and letting the work anew on the unit-price basis. Not only has this been the means of saving a great deal on overhead charges, but it is gratifying to know that much lower prices are being secured on the unit-price basis than if the work had been continued on the cost-plus basis.

In conclusion, the speaker desires to express his gratification at having had the opportunity of being connected with the cantonment work—a feeling which he is sure will grow as time goes on and as the troubles connected with the work are replaced in his memory by the more pressing ones of other work.

Mr. Dunham.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—An important item in this interesting paper is the allowance of 50 gal. per capita. Every superintendent and manager of water-works has estimated the reduction in waste if he could have his consumers placed under military discipline. All have some knowledge of the subject. Figures from the average city are misleading, because they are obtained by using the population as a factor, whereas many people may obtain supplies from deep wells or other sources. A correction would increase the per capita allowance, but the cantonment figures are definite. Although a few cities like Fall River, Mass., and Providence, R. I., could have been cited as object lessons, courage was necessary in order to estimate as low as 50 gal.

The reply to one question—which may not be a fair one—would be of great interest: With the experience thus far gained, would the author, on a similar demand for advice to the Government, recommend the same per capita allowance?

It would also be interesting to know to what extent the force-main capacities were determined by fire-protection needs, and whether extra high pressure at the pumps was used in order to provide for the domestic supply during such a short period as a fire might continue.

* New York City.

Mr. Johnson.

GEORGE A. JOHNSON,* M. AM. SOC. C. E. (by letter).—The highly important functions performed by Col. Maury in connection with the great work of the Construction Division of the Army, makes noteworthy his becoming modesty in describing his participation in the unparalleled achievements of the Division and the happy manner in which he shares his very creditable record with his associates. It appears to be pretty well recognized at this time that the Construction Division as a whole accomplished a feat unequalled in the building history of the world, and, in this performance, Col. Maury is clearly correct in the premise that there is ample credit and glory for all who had a hand in it.

The rank and file of the Division, in addition to the material work they accomplished, established still another record of quite a different, and perhaps of quite as noteworthy, a character. As paper advertisers of their Division they made a sorry record, and as seekers after the plaudits of the public, they were almost a total loss. Probably no such aggregation of "go-getters" and "do-it-nows" was ever before brought together, and the conspicuous absence of endeavors to secure personal benefit from the work they were doing in the great emergency redounds to the credit of the organization as substantially proving that they were laboring unselfishly to serve their country first, last, and all the time.

On reading Col. Maury's paper one may at first thought conclude that with such utterly unprecedented methods of providing water supply facilities in the camps and cantonments as were of necessity followed in 1917, inadequacies, inapplicable features of importance, and structural inferiorities would make the road of the operating man who came after the intensive builder, a rocky one indeed. The writer was that man, and when he, like Col. Maury, was wired into the service (not by the time-consuming telegraphic process applied in Col. Maury's case, but by the quick acting telephone), he contemplatively reviewed on the train the probable formidable array of difficulties to be overcome because of the very nature of the problem, and the obstacles to success which he might reasonably expect to encounter, due solely to the speed with which the various systems had been put together.

Before he was made Assistant to Col. C. D. Hartman, Officer in Charge of the Maintenance and Repair Division, the writer was in direct charge of the Water and Sewage Section thereof, and, as such, exercised supervisory control over the water supplies in about 400 military establishments. In the permanent establishments the population to be served increased materially, following the declaration of war, of course, and, in each case, provision for adequate and pure water supplies had to be made, in order to keep step with the requirements, and a little ahead of them. In the camps and cantonments,

* New York City.

Mr. Johnson.

although the problem was one involving entirely new construction, two factors introducing important complications had to be faced and solved satisfactorily, namely, to get the systems functioning on time and to provide, so far as was humanly possible, for the fluctuating population.

It is remarkable at this date to review the events following September, 1917, when the first increments of the new army began to arrive at the camps. The water systems were made ready in time; the supply was sufficient and pure; and fire protection well cared for by ample water pressures. All the water systems had been designed on a basis of a certain fixed estimated number of troops and animals at each individual camp, and a factor of safety of 2.85 times the average demand allowed in order that peak load consumptions might be anticipated satisfactorily.

The rapidly fluctuating camp population was perhaps the most difficult problem for the operator to meet, and a matter which the designers of the water systems could not anticipate with precision. A notable case in this line was Camp Dix which was laid out in all its parts for a maximum population of less than 45 000. At 3 P. M., on one day, the population was 44 000. At 10.00 A. M., the day following, it was 60 000. The population of a small city had been moved in over night. How many city water superintendents have ever had such a problem to meet? With the facilities provided by the designers and the exercise of considerable resourcefulness on the part of the Utilities Officer, however, every man was adequately supplied with water, and fire protection requirements were met.

The water systems at all the military establishments were equal to the demands made on them. Naturally, in some cases, additional storage had to be provided, more pumping capacity, and similar augmentations made in the original systems, but the outstanding fact is that, as a whole, these water systems, conceived over night as it were (a fact confirmed by Col. Maury's description of the work at Camp Lewis), built in a mere fraction of the time usually required for such work, and called on to function satisfactorily from the start, did what they were expected to do. It was little short of a miracle that they did.

At the close of 1918, the Maintenance and Repair Division of the Construction Division of the Army had charge of the operation and maintenance of the water supply, sewerage, electrical, heating, refrigeration, and fire-fighting plants and systems, and the maintenance of the buildings and roads, in 46 camps and cantonments, 51 hospitals, 66 border stations, 25 supply depots and terminals, and 271 regular military posts, and, for its uses during the fiscal year ending June 30th, 1919, funds to the amount of $62 400 000 were allotted. Its field force, concerned solely in such operation and maintenance work at these points, numbered 452 commissioned officers and 16 559 enlisted men.

Mr. Johnson.

In the head office in Washington, there were 20 commissioned officers and 126 civilians at the close of 1918.

At each completed project, in charge of the operating force, there is an officer selected on his record in civil life in the management of public utilities, holding a position comparable to that of city manager, but with detailed responsibilities and duties far transcending those of such an official. Indeed, he is a composite of city manager, city engineer, fire marshal, street and building inspector, manager of municipal water-works and sewers, telephone and electric supply companies, and cold storage, besides being general all-around handy-man to whom everybody comes to get anything fixed.

The local field organization is military in its character, and was worked out with a view to obtaining the greatest efficiency and the best results with a minimum of personnel and expense. A major is usually placed in charge of this highly important work, and has as assistants a number of commissioned officers of lower rank. For operative and clerical assistants, there is a detachment of enlisted men, the number depending on the size of the project, but varying from 300 to 800 in various camps and cantonments.

Each utilities officer maintains in his office a "trouble department" —a desk over which passes every complaint of whatever nature and every request for repairs and replacements. In addition to the written communications received, the number of telephone calls in the trouble department at each camp averages several hundred per day. All the work is supervised, co-ordinated, and administered from the Washington office. Refinements and economies are constantly worked out, and their effect is always toward giving the greatest service with a minimum expenditure of public funds. The Maintenance and Repair Division of the Construction Division's office is really a huge service organization, operating for the benefit of more than a million persons in several hundred locations, and designated to supply them with every convenience and comfort incident to their existence as soldiers.

Failure in the great task delegated to the Construction Division of the Army was freely predicted, but the seemingly impossible was done effectually and economically. No greater tribute can be paid to those whose ability, untiring energy, and unselfish devotion were directly responsible for the successful performance of the colossal task of building the camps and cantonments in which to train the new army, and the other facilities required to make ready its translation to French soil, than to say that they were not found wanting in the supreme undertaking to which the nation had dedicated itself. Not only this, they have managed uniformly the hundreds of military establishments built by them in a fashion calculated to inspire improvement in existing methods of doing work along more or less parallel lines in civil life.

F. R. Harris,* M. Am. Soc. C. E. (by letter).—Although this paper necessarily is of such a brief and general character that what the writer is about to state can hardly be considered in the nature of a discussion of its technical details, nevertheless, he would like to add his praise to the high character of the work performed by the Construction Division of the Army, and especially to the very excellent results produced by Col. Maury, as Advisory Engineer in general charge of the water supply development in connection with the cantonments and other kindred work. It was certainly of the greatest benefit to the country that the services of such able experts as the author and others whom he has mentioned as having assisted him, were made available to the Government. Mr. Harris.

During the early part of the war, the strenuous efforts of the United States to expedite its military programme resulted in the concentration of work in various locations along the Atlantic seaboard. In some places this concentration resulted in congestion and overcrowding so that in the smaller communities what are known as public utilities were thoroughly incapable of meeting the demands of the military and civil population congregating around these centers. This was especially so in and around the Port of Norfolk (using the word "Norfolk" as a designation to embrace the Port of Hampton Roads). The Port of New York was being used to its greatest capacity. It was so greatly developed and had been built up so normally that it was not susceptible of great expansion. The port facilities which Hampton Roads offered, were naturally great, but undeveloped.

On the Newport News side, the War Department had undertaken the construction of general embarkation camps, animal embarkation camps, corrals, storehouses, hospitals, and various other necessary works. On the Norfolk side, it started a huge new Army Supply Base, took over the terminals of the Norfolk and Western Railway, and proceeded to develop and enlarge these for an Engineers' Material Embarkation Depot. At Pig Point, near the mouth of the Nansemond River, it planned a large Ordnance Embarkation Station; later, at Camp Abram Eustis, on the Newport News side, it constructed one of the largest of the artillery encampments and proving grounds.

The Navy Department had well under way the plans for enlarging and rebuilding the Norfolk Navy Yard, the construction at Sewall's Point, on Hampton Roads, of a Naval Operating Base, a great extension of the Naval Magazine, and, later, the construction, in the vicinity of Yorktown, of a Fleet Fuel Oil Reserve Depot and Ordnance Station. Directly to the north of this, the War Department, under contract with the du Ponts, planned a huge shell-loading plant at Penniman. The Navy Department and Shipping Board jointly under-

* Rear-Admiral, C. E. C., U. S. N., Norfolk, Va.

Mr. Harris.

took great extensions in the Newport News Shipbuilding Plant, and, incidentally, the various Departments interested took over for operation in connection with the Railroad Administration the large coaling plants of the Chesapeake and Ohio, Norfolk and Western, and Virginian Railroads.

All these construction activities starting at or about the same time, and entailing together a cost of approximately $200 000 000 taxed this community far beyond its capacity to furnish the requisite labor, house the labor furnished, and, considering the military personnel on the Newport News side, amounting usually to about 50 000, and the naval personnel on the Norfolk side, amounting probably to 20 000 men, resulted in practically a complete breakdown of all community utilities such as transportation, electric power and light, roads, water supply, etc.

In the earliest days of this problem, Col. Maury, acting for the Construction Division of the Army, attempted the task of expanding and extending the water supply in the vicinity of the Port of Embarkation at Newport News, and began to study the problem of the Norfolk water supply. Practically, single-handed, he obtained much information for future use and did much good on the Newport News side within the limited funds and authority at his disposal.

The conditions of congestion previously enumerated resulted in the inevitable. None of the work on the programme could be pushed, there were insufficient men, materials, and resources, to push all of them at once, and as there was no priority as to which should be done first and no organization to plan for systematizing the work, orders for which had been issued by several different Departments and Bureaus acting independently of one another, congestion, waste, and a most serious labor turn-over resulted. The two military Departments, War and Navy, finally attempted to correct the difficulties by appointing a representative of each Department to confer and agree on priorities, etc. Later, this was expanded into what was known as the Board of Control, War Construction Activities, in which the War and Navy Departments, the Department of Labor, U. S. Housing Corporation, U. S. Employment Service, Railroad Administration, Fuel Administration, Shipping Board, Emergency Fleet Corporation, and, later, the War Industries Board, were represented, of which Board the writer became President. The agreement as to the organization of this Board and its scope and authority was subscribed to by the Secretaries or heads of these various Departments, and an attempt was made through it to co-ordinate the efforts in the Hampton Roads District for the general good, with the idea of advancing the war construction programme. This Board soon gave its attention to the conditions of congestion and to the inadequacy of the utilities serving the port. Funds were secured through the instrumentality of

the Chief Executive from the various Government Departments interested, and allotted to the performance and securing of adequate water supply, transportation, light and power, roads, etc. The writer then first became acquainted with Col. Maury, and was guided and advised by him relative to the needs for water supply extension in the District. Col. Maury also played a large part in the plans and programme finally decided on and approved for securing potable water to meet the demands of the Port of Norfolk. Mr. Harris.

In this preliminary work, it is only fair to state that the services of Maj. J. A. Bensel, Corps of Engineers, U. S. A., Past-President, Am. Soc. C. E., who had been assigned to the Board as an assistant, contributed largely and, in fact, first brought to the Board's attention the conditions with respect to the water supply and the absolute need of taking immediate steps to secure its extension and enlargement, as an alternative to seriously hampering the War Department's plans with respect to the use of the port for the embarkation of the assigned quota of military forces to be transported to France.

Col. Maury impressed the writer at the conference at Washington so favorably with his thorough knowledge of local conditions, and his familiarity with what was needed, that, in the allocation of the improvement in water supply of the port, the writer advocated that this work be entrusted to the Construction Division of the Army, with the understanding that Col. Maury was to give it his general supervision. Under that arrangement Col. Maury took up the negotiations and planned the work for the extension of the Newport News water supply, thus practically trebling its original capacity, the extension of the Portsmouth water supply on about the same basis, and a co-ordination of the two separate water supplies of the City of Norfolk, with a view to placing them under one management. All this work, which is now completed, or rapidly nearing completion, although not carried out to its fullest extent on account of the end of the war, has involved an expenditure of upward of $5 000 000. Considering the limited time and the various other difficulties which the writer has not time to enumerate, but which are common to war construction activities undertaken in times of such great stress, the work has been well carried out to the great credit of Col. Maury and the organization which he represented.

The writer desires principally to state, as a matter of record, his appreciation of Col. Maury's zeal, untiring effort, and willingness, under the most adverse conditions, to carry out the very great public service that he performed.

Maurice R. Scharff,* M. Am. Soc. C. E. (by letter).—As one who worked, as far as water supply was concerned, under the supervision of Col. Maury in the stirring days of July to October, 1917, Mr. Scharff.

* Pittsburgh, Pa.

Mr. Scharff.

the writer desires to express his appreciation of having had the opportunity to take part in the fine work of the Cantonment Construction Division, and the pleasure with which he will always remember the helpful attitude of the Washington office. Perhaps its greatest helpfulness lay in the fact that the plan of operation, as described in the paper, was not always strictly enforced and that a large measure of freedom was left to the Supervising Engineers in planning and constructing the work, while the supervision of the Washington office was confined to the enforcement of general standards and to help in securing rapid deliveries of materials.

The conditions under which the writer served as Deputy in Charge for Morris Knowles Incorporated, Supervising Engineers at Camp McClellan, Anniston, Ala., illustrate very well the impossibility of submitting plans and reports to Washington and awaiting decisions and preparation of bills of material in the Washington office. When the writer arrived on the ground on July 16th, 1917, the site for the camp was not yet selected, although arrangements had been completed by the local Chamber of Commerce for the purchase of a large tract of property which offered a number of possible locations for a divisional camp. Within the ensuing two weeks, the site was selected, the general plan of the camp outlined, railroad connections arranged for, and construction on the first unit started by the contractor. During the same time, a decision was reached to secure a water supply by connection to the mains of the water company supplying the City of Anniston, by the construction of a booster station and reinforcing and supply mains, 32 000 ft. in length. About August 1st, word was received that water must be made available in the camp for the troops who would commence to arrive on August 15th. The delivery and distribution of the pipe commenced the same day; pipe-laying started on August 4th; and although two days of rain intervened, on which no pipe was laid, the line was tied in and an ample supply of water for the first troops delivered to the camp by August 15th.

Some further facts about this water supply system are of possible interest, as illustrating the extent to which the design and construction were controlled by conditions other than those which would apply in ordinary practice. The source of supply of the water company supplying the City of Anniston is Coldwater Spring, a great spring gushing out of the limestone and said to have a flow of 25 000 000 gal. per day. Water taken from this spring is pumped to a distributing reservoir at such an elevation as to give a static head at the camp site of more than 50 lb. per sq. in. The largest size of pipe which there was any possibility of obtaining within the requisite time, however, was 10 in. in diameter, and this size was selected on the basis of possible deliveries, without regard for the fact that theoretical computations appeared to indicate that a larger size might have been more

Mr. Scharff.

economical. With this size of pipe, it was estimated that at the assumed maximum daily rate of draft —1 500 000 gal.— a pressure of not less than 30 lb. per sq. in. could be maintained at the site selected for the location of a booster station, which would be sufficient to maintain domestic service to the sections of the city and suburbs dependent on this portion of the distribution system; and that by raising the pressure at the booster station to 130 lb. per sq. in., this quantity could be delivered to the camp with a resultant pressure at the camp site of 50 lb. per sq. in.

Another feature of the water-works system at Camp McClellan, which was controlled entirely by the time limit set for completion, was the location of the 10-in. supply main which followed, for the greater portion of its length, the existing tracks of the Southern Railway and the connecting line constructed for the service of the divisional camp. A location approximately 4 500 ft. shorter would have been available by following some of the existing country roads, but as these roads were practically impassable for heavy traffic, it would have been impossible to complete the delivery of the pipe within the time limit fixed by military necessity.

The rapid delivery of the pipe required by this 32 000-ft. supply main and its completion and connection within 10 working days, must be ascribed largely to the authorization by the Washington office of the use of cast-iron pipe and to the co-operation of the Superintendent of the Anniston foundry of the United States Cast Iron Pipe and Foundry Company. This company furnished from its Anniston, Chattanooga, and Bessemer foundries all the pipe in the line, except about 5 000 ft. of Universal pipe, and when completed, the line contained in addition to the Universal pipe, standard bell and spigot pipe of Classes A, B, C and D; standard gas pipe of two weights; and several thousand feet of 9.8-in. (25-cm.) pipe of special quality, which had been cast for the Argentine Republic. Not only did the foundry superintendent devote a large part of his time in securing these deliveries, but, in addition, he arranged for the use of the foundry pipe crane, much of the time under his personal supervision, to distribute more than 20 000 ft. of pipe along the trench from the railroad, with maximum rapidity and minimum breakage.

The service of the Anniston foundry was also of the greatest value in securing the rapid delivery of special castings and even of flanged castings. On one occasion, while additional pumps were being constructed in the Coldwater pumping station of the water company, a crack was discovered on a Sunday afternoon in a 16 by 10-in. reducer. The superintendent of the foundry was reached on the telephone, and, on Monday morning, the completed casting, faced and drilled and still hot from the moulds, was delivered at the pumping station over

Mr. Scharff.

8 miles of country road. This service permitted the casting to be placed in position and pressure restored by 4 P. M. on Monday.

The early operations of the water-works system at Camp McClellan produced interesting examples of the difficulties, due to the peculiar character of water consumption in military encampments. The pressure from the city distributing reservoir was sufficient to deliver the quantity necessary for night use and the requirements of the first troops which arrived on August 15th, without booster pumping; but after the completion of the construction of the first booster pump, on August 19th, it was necessary to operate the booster pumping station throughout the day, every day, from about 5 A. M. to 7 or 8 P. M. This method of operation was sufficient to permit the continuous maintenance of satisfactory service after the completion of the camp distributing reservoir on October 13th; but, until that time, much difficulty was experienced in maintaining service, due to rapid fluctuations in pressure resulting from the large variations in hourly use. Prior to the completion of the reservoir, it was found to be impossible to maintain pressure in the higher parts of the camps immediately after the noon mess hour, when the mess kits were being cleaned, and for the greater part of Wednesday and Saturday afternoons when there was no drill and when most of the showers in the camp would be found running either for bathing or washing purposes. These difficulties were especially great during the period from August 19th to September 11th, when the booster station was pumping into a closed system, not only without storage, but without adequate pressure relief. During this period, a direct telephone connection was maintained between the booster pumping station and the engineering office at the camp, and during certain portions of the day, an observer was stationed at each point watching the pressure gauges, the observer at the camp directing the operation of the suction valve on the pump so as to control the pressure at the camp. In spite of this precaution sudden pressure jumps occurred on several occasions, and breaks in the supply main resulted once or twice.

On September 11th, a pressure regulating stand-pipe of 10-in. pipe supported by a wooden scaffolding, was constructed and carried up to the elevation of the distributing reservoir (then under construction). At the top of this stand-pipe, an elbow and a 6-in. valve were leaded in, and this overflow at the highest point of the system proved a reasonably effective pressure regulator, although at times, some attention to the pressure gauge at the camp and control by regulation of the suction valves at the booster station were necessary.

In some respects, there was much similarity between the cantonment water supply work and that on which the writer was subsequently engaged in the Water Supply Division of the Office of the Director of Construction and Forestry of the American Expeditionary Forces in

France. In both cases, the time element was of far greater importance than the ordinary considerations controlling engineering design. In one respect, however, the cantonment construction work was very much more satisfactory and enjoyable than that in France. Reference is made to the availability and orderly rapid delivery of material made possible by the efficient co-operation of the Washington office, as compared with the discouraging impossibility of securing materials, which resulted from the tonnage shortage during the early days of the history of the American Expeditionary Forces. Mr. Scharff.

SAMUEL A. GREELEY,* M. AM. SOC. C. E. (by letter).—All who served as supervising engineers for the construction of the National Army camps, will feel grateful to Col. Maury for his excellent and generous paper on the water supply work of the Construction Division. The writer's work was at Camp Custer, Mich., in 1917, as Consulting Engineer, in charge of the water supply and sewerage, and, in 1918, as Supervising Engineer for a 40% extension to the camp, which brought its total capacity to about 50 000 troops. It is thought that some additional data having particular reference to construction and operation will prove helpful, with a reference to several more detailed descriptions. The writer's connection with both construction periods enabled him to acquire a perspective after the first rushing season of 1917 and prior to the second period which began in August, 1918. Mr. Greeley.

He first visited the site of Camp Custer on June 12th, 1917, and several days later sent a preliminary report to Washington covering the water supply, sewerage, highways, and railroad connections. On June 29th, he went to Washington for a conference. The helpfulness of the Advisory Engineer for Water Supply and his staff was thus early discovered, and thereafter this assistance was sought from time to time through conference and by correspondence.

Organization.—It was realized during the 1917 experience that a clear-cut organization with definite responsibilities was of paramount importance in expediting the work. This organization was definitely accomplished in 1918. The office of Executive Engineer was especially helpful, as it handled all office details, including matters relating to personnel, reports, weekly meetings, transportation, and the like. At the start each division engineer was given definite instructions relative to keeping permanent records of the work as built, cost records, and data for reports to Washington. This all helped greatly in finally winding up the work.

Progress.—It was also found that complete up-to-the-minute progress records were essential to finishing the work on scheduled time and to maintaining pressure where it was most needed. A special progress department of three engineers was formed. The duty of this depart-

* Chicago, Ill.

Mr. Greeley.

ment was primarily to receive, tabulate, and correlate the daily progress records of the various divisions. Daily reports were submitted by the divisions to the Progress Engineer, which he charted for the Constructing Quartermaster and the Supervising Engineer. From these charts, weekly reports were prepared for the Washington office.

The analysis of progress from day to day was based on time-labor rather than on cost, and cost summaries were carried separately. This was done to meet the war demand for rapid work. The estimated relation between time-labor and cost for the various divisions of the work, as used on the Camp Custer Extension (1918), is given in Table 2.

TABLE 2.

Division.	Percentage of Total Work:	
	Time-Labor Basis.	Cost Basis.
Buildings	63	77.4
Water supply	12	9.5
Sewerage	10	3.1
Roads	7	3.35
Railroads	5	3.35
Outside electrical	3	9.3
Total	100.0	100.0

A daily general progress chart was posted, showing the essential elements. The linear feet of water pipe, sewer pipe, and concrete road built each day showed the future rate of progress necessary to completion by a given date and assisted in determining the number of trenching machines needed, etc. The delivery of materials also influenced this feature.

The Progress Engineer was also called on to keep in touch with the inspection of the work, so as to afford an outside disinterested check on each division as to progress and workmanship.

Sanitation.—Col. Maury mentions the epidemic of influenza as one of several items which combined to increase the cost of the work. The extension to Camp Custer was started just as the influenza began to develop in the Middle West. To reduce it as much as possible, and to maintain the construction camp in a clean, healthy condition, a division of sanitation was organized. The results were gratifying. From October 6th, 1918, when the working force was 1 281, to November 15th, 1918, when it was 2 572, there were 4 cases of influenza and 1 case of pneumonia, and altogether only 14 cases of sickness requiring the attention of a physician. At this time, 65% of the work was completed. From November 15th to December 31st, 1918, when the working force fell

from 2 572 to 1 080, and 90% of the work had been done, only 10 more cases of sickness occurred. This was accomplished by active attention to all the details of sanitation, including water supply, bunk houses, stables, restaurants, cleaning of grounds, personal cleanliness, etc. In connection with the water supply, a quickly set-up drinking fountain, of which fifty were placed about the work, was used. During the active period of construction the sanitary squad comprised 80 men, of whom 23 were on night duty. It is believed that these results not only maintained a good working force, but also increased applications for work as the healthfulness of the construction camp was well known in labor circles throughout Michigan and the adjoining States. Mr. Greeley.

Construction.—The water distribution system was built almost entirely of wood pipe. In 1917, the bells of all cast-iron fittings were ground to fit the tenons of the wood pipe. During 1918, standard fittings with lead rings were used. The bulk of the trenching was done by machine, the average rate of progress in 1918 being 342 ft. per machine day. The work was slowed up by the difficulty of securing cast-iron fittings. On the completion of a section of the distribution system, the pipes were flushed and "hypo" was pumped in until bacterial analyses showed the water to be safe.

At the pumping station, the principal feature of the 1918 work was the erection and cutting in of new units without interrupting the service. This was done with a total shut-down of 53 min., during which time the camp was supplied from storage. Two motor-driven and one gasoline-engine-driven centrifugal pumps were installed and connected to the wells built in 1917. As nearly as could be ascertained, the cost of 8 and 10-in. wood mains in 1918 was $2 per ft.

Operation.—One of the satisfactory features of the camp work was the opportunity of putting the works into operation. At the close of the 1917 construction period, tests of the centrifugal pumps were made and characteristic curves drawn. An operating outline was thus determined. In addition, a chart was prepared for recording daily the camp population, the coal consumption at heating plants, and the water consumption daily and per capita. A study of fixture losses in lavatories was made, and a preliminary list of items for the regulation of consumption was prepared.

Summary.—The writer believes that the construction of the camps was well done, all things being considered, and that one of the outstanding features was the use of the country's engineering resources, marshalled and directed through the Washington office, as outlined by Col. Maury. No serious errors have appeared.

Lynn E. Perry,* Assoc. M. Am. Soc. C. E. (by letter).—The Society is much indebted to Col. Maury for this valuable paper. Indeed, the Mr. Perry.

* Easton, Pa.

Mr. Perry.

country is indebted to him and his capable and patriotic corps of assistants comprising the Construction Division of the Quartermaster Corps.

Considerable courage was needed to adopt so low a per capita daily consumption of water as 55 gal. The writer is positive that not many data were available concerning the water consumption of a soldier as contradistinctive to a civilian; and it could only have been Col. Maury's extremely broad experience and ripe judgment that led him to adopt so economical and, at the same time, so adequate a figure.

The writer's experience with the water problem at the large cantonments in the United States has been that of consumer only. He was, however, for a short time associated with the Aviation Section of the Signal Corps at one of the fields under construction. This field was designed for six aero squadrons and proportionate medical and quartermaster detachments, a total of about 1 018 officers and men. The maximum population during the writer's stay was 1 066.

The water supply was lifted from four wells, about 300 ft. deep, into a concrete reservoir; thence, it was pumped to a tank 135 ft. high. Aviation fields are usually located on fairly level sites, and this field was no exception. A geological investigation indicated that the water entered the stratum at an outcrop about 40 miles north of the field; the quality was good.

During construction the engineer was persuaded to sink one well to a lower stratum, the quality of the water from which was known to be excellent and was being used by a near-by large city. The well, however, would yield only about 67 gal. per min. and was drawn to the shallower stratum, the orders being to sink four wells each yielding 100 gal. per min.

Thus, four wells were provided, each yielding 100 gal. per min., or a total capacity of more than 500 000 gal. per day, for a population of about 1 000. Either well would supply 140 gal. per capita per day which, incidentally, was about the consumption. The entire system was constructed of cast iron and the tank was made of steel. Although the writer advised an emergency chlorinator, none was provided during his stay. During construction, when an inferior quality of water was being used, an emergency hypochlorite plant had to be provided.

The writer's attention was particularly drawn to the sanitary precautions mentioned in the paper. He has always been an enthusiastic advocate of emergency chlorinators, especially where the water is of questionable quality, and believes that these were finally provided at all the cantonments and camps in this country.

The covering of reservoirs and tanks to prevent pollution and the growth of algæ is also very important. The writer had considerable difficulty in obtaining materials for this purpose in France; at least

one reservoir, that for the Kerhuon Hospital Center, near Brest, was covered with canvas, after having been in use for about one year. Mr. Perry.

It would be instructive and interesting to have a discussion from some one connected with the Construction Division of the Signal Corps, in connection with the experience and activities of that body during recent hostilities. A paper presenting the work of the Water Service in France would also be timely, and the writer trusts that such a paper will be written.

Asa E. Phillips,* M. Am. Soc. C. E. (by letter).—In the discussion of this admirable paper on water supply for camps and cantonments, which only briefly outlines the great work done by Col. Maury and his staff, there should be recorded, with just estimate of its paramount value, the remarkable rapidity with which results were achieved in completing and putting into operation safe, adequate and reliable water supply systems for these extraordinary military establishments, which results could only have been accomplished by rare good judgment, reinforced by instant decision and exceptional foresight. In reading this paper one gets only a glimpse between the lines of these qualities on which the success of this great work depended so completely. Mr. Phillips.

Perhaps one of the war expressions that will linger longest in the minds of many is "quantity production"—so often a Utopia not reached until after Armistice Day. It should be recorded, however, in the annals of Engineering, that of all war activities none approached the great achievement of the Construction Division of the War Department in instantly striding into quantity production from the day the organization was set up; and this could only have been due to the extraordinary quality of the personnel of the commissioned officers on Gen. Marshall's staff. In the remarkable results achieved, Col. Maury contributed much, and that was of vital significance.

In many ways this work on the water supply for cantonments and camps is of permanent value. The wide experience it has given in the use of wood-stave pipe, in standards of lay-out and construction, in safeguarding water supply, and enlightening work in the way of fire protection, has left the leaven of sound practice in the far reaches of the country.

Thus far, much of the discussion on this paper has been devoted to consideration of the unit allowance of 50 gal. per capita per day, and, occasionally, there occurs a lament at the average per capita daily consumption of the larger cities. "One of the greatest economic municipal crimes of the United States", says P. H. Norcross, M. Am. Soc. C. E., in his discussion, "is the extravagant use of water. In no city is this emphasized more acutely than in Washington, D. C., where the

* Washington, D. C.

Mr. Phillips.

daily consumption runs as high as 150 to 160 gal. per capita, and where practically 60% of the total water furnished is used by Government Departments, for which no direct charge is made. The remaining 40%, which is used by the private consumer, bears the entire burden."

There is much truth in what Mr. Norcross says, and yet his data set forth the situation so imperfectly that it may be well to record, as accurately as is now possible, some figures as to water consumption at Washington. The average daily consumption for the fiscal year ending June 30th, 1919, was 64 428 356 gal. distributed about as follows:

Domestic (dwellings and apartments), 88% metered	20 000 000 gal.
Large private consumers, all metered	12 000 000 "
Municipal buildings and works	3 500 000 "
U. S. Government	25 000 000 "
Unaccounted for	3 900 000 "

From this it will be seen that the domestic consumption is 31% of the total, large consumers 19%, municipal 5%, and the U. S. Government, 40 per cent. The domestic consumption is about 45 gal. per capita, and the total consumption about 142 gal. per capita.

The total revenue from water rates for this fiscal year was $782 159.36, derived about as follows:

Domestic (dwellings and apartments)	$560 000.00
Large consumers	222 000.00
Municipal	None
U. S. Government	None

Thus, it is seen that the domestic consumers pay 72% of the total tax for 31% of the water; the large consumers pay 28% of the tax for 19% of the water; and the United States pays nothing for 40% of the water.

Now, it is quite apparent from these figures that Washington can lay no claim to pre-eminence in the category of "municipal crimes" of this character. Whatever there may be of undue usage of water, it certainly cannot be charged to the domestic consumer, who modestly pays three-quarters of the tax and receives one-third of the water, or to the municipality.

With this unbalanced adjustment, if it were not for the revenue derived from the large consumers, sufficient funds would not be available to operate the Water Department, and, perhaps, this condition has permitted somewhat greater liberality as to usage by the large private consumer, the only other source of revenue, since the United States declines to pay anything. As a tax on municipal use must come from the taxpayer, and only means taking money out of one pocket to put it into another and but little increase in revenue would be had in this

way, it presents small opportunity for relief, although doubtless sound in principle. Mr. Phillips.

The only large manufacturing establishments in Washington belong to the United States Government. Of these, the Naval Gun Factory, the Government Printing Office, and the Bureau of Engraving and Printing use about 40% of the total Government consumption, the remainder being used in other Federal establishments, such as hospitals, laboratories, and offices. Much of this usage may well be considered wasteful, but the total must inevitably be large.

Probably no other city in the country has a more evenly distributed domestic consumption throughout its population, so that the average daily domestic consumption must be considered to be quite low, when one takes into account the character of the population, as well as the physical characteristics of the National capital.

Dabney H. Maury,* M. Am. Soc. C. E. (by letter).—Mr. Dunham asks whether, with the experience thus far gained, the writer would, on a similar demand for advice to the Government, recommend the same per capita allowance. To this the writer would unhesitatingly reply that, under similar conditions, he would. This answer should not, however, be construed to apply without exception to all sorts of Government activities, in peace as well as in war. Mr. Maury.

In time of peace, restrictive regulations and military discipline generally are apt to be less rigidly enforced than in time of war. Furthermore, in time of peace it is exceedingly unlikely that any army post would be garrisoned to its full capacity, and it was observed in every case that as the number of men at any camp was reduced, the per capita consumption increased, and *vice versa*.

The capacity of the water system should be great enough to give each man his proper daily allowance, assuming the camp to be fully occupied. Then, if a large number of men are ordered away, it is to be expected that the per capita consumption will increase, because all the leakage and part of the waste will continue as before; but there will still be enough water for the men remaining, and the increase in per capita consumption will not have nearly as bad an effect as would be the case if it resulted in an actual shortage of water.

It was the writer's observation that, as a rule, the per capita consumption at the older and permanent posts was extravagantly high, often several hundred gallons per day. This was due in part to the large quantities of water used for lawn sprinkling; in part to seriously defective plumbing, especially in Coast Defense Fortifications, where brackish water had caused many leaks; and in part to waste of water in the officers' quarters.

* Chicago, Ill.

Mr. Maury.

What a man gets for nothing, he neither appreciates nor conserves. If all the officers' quarters were metered, with water furnished free to the amount of, say, 50 gal. per day for each occupant of the quarters, and a good stiff meter rate added for all water used in excess of that allowance, much could be accomplished in the way of reducing the consumption at these army posts.

The writer is well aware, however, that the enforcement of this regulation would add nothing to the popularity of the already much-abused Utility Officer, and that for a few days after rendering his monthly water bills, he might feel tempted to absent himself from the officers' mess and club.

In reply to Mr. Dunham's second question, the writer would state that the water-works systems of the cantonments were designed to furnish, in addition to one-quarter of the peak rate of domestic consumption for a fully occupied camp, water for fire purposes at the rate of 2 000 gal. per min., at a net pressure of not less than 50 lb. per sq. in., at any group of hydrants on the distribution system. The fire-drill regulations made it the definite duty of certain designated individuals to shut off, as soon as the fire alarm was sounded, every fixture from which water might be drawn for purposes other than fire-fighting, except those necessary for cooking purposes. There were two principal mains, 480 ft. apart, running through each regimental unit, with hydrants spaced only 215 ft. apart on each main, so that twelve streams could be concentrated on any building between the mains, without laying any line of hose longer than 350 ft. The shortness of the lines of hose and the use of small hose nozzles, only $\frac{3}{4}$ in. in diameter, combined to give very small friction losses through the hose. There was good separation of buildings—40 ft. between two-story buildings, and 30 ft. between one-story buildings or between a one-story and a two-story building. There were also fire gaps of 300 ft. between brigade units or groups of buildings. Plenty of men were available, of course, to handle the hose and the hydrants.

The net effect of all these specifications was that any building could be attacked from many different directions by a large number of fire streams, with good nozzle pressure and good horizontal projection of stream. The results obtained in extinguishing fires and in keeping down fire losses were summarized in the paper.

Mr. J. Waldo Smith asks whether the writer would recommend the form of contract used in building the cantonments for general adoption in time of peace.

That form of contract was unquestionably the only one under which the cantonments could have been constructed within the time allowed. Its development was almost a stroke of genius on the part of the members of the Advisory Committee referred to by the writer; and this is especially true if genius be defined as "an infinite capacity

for taking pains", for that contract represented weeks of the most careful study by some of the brightest minds of the Profession. Mr. Maury.

The writer, however, concurs in Mr. Smith's opinion that the cost-plus-a-fixed-fee contract is not one which should be adopted generally for all construction in time of peace, and especially where speed is not of vital importance.

In conclusion, the writer fully realizes his good fortune in having had his paper discussed by so many of his personal friends, and he would again remind his readers that the design and construction of the water-works for the army activities were in no sense a one-man job, and that the credit for such results as were obtained should be shared by all the many men who had a hand in the work.

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AMERICAN HIGHWAYS

AN INFORMAL DISCUSSION AT A MEETING OF THE SOCIETY, APRIL 16TH, 1919.

BY MESSRS. W. G. B. THOMPSON, H. ELTINGE BREED, EUGENE W. STERN, CALVIN TOMKINS, H. F. DUNHAM, AND WILLIAM E. WOOLLEY.

Mr. Thompson.

W. G. B. THOMPSON,* ASSOC. M. AM. SOC. C. E.—The population of what is generally known as the metropolitan district of New York is, at present, approximately 7 000 000 people, and is rapidly increasing in volume and density. This district may be said to embrace Brooklyn, Queens, Richmond, Manhattan, the Bronx, and the territory of New Jersey lying between the Hudson and Passaic Rivers.

Approximately, 30 000 miles of railroads have their termini on the New Jersey side, and seventy or more steamship lines operate from the joint port to all parts of the globe. Traffic and freight transfer conditions are daily becoming more difficult to handle, economically and expeditiously, and, unless an immediate and serious effort is made to devise a general plan or scheme which will co-ordinate all the factors in the problem in a manner to afford early relief, with ample provision for indefinite future expansion, not only the supremacy of the Port of New York will be endangered, but the very existence of its people (especially on Manhattan Island), may conceivably be threatened. This was exemplified during the winter of 1917-18 when severe weather and ice prevented coal movement by water to the city, and during the recent strikes of harbor boatmen, when coal and food supplies were threatened, and thousands of workers were unable to proceed to their offices and shops, due to the ferry systems being tied up.

It will be especially interesting to note the effect on the daily supply and life of the city in connection with the general harbor strike called

* Trenton, N. J.

Mr. Thompson.

for April 15th, 1919, when 16 000 men may quit work because of differences with employers. According to the plans of the strike leaders, not a single exception is to be made in favor of any class of harbor activity. The leaders claim that unionists will quit Army and Navy craft, except where manned by enlisted service men. They will also quit all railroad ferries and, it is reported, will refuse aid in berthing hospital ships from abroad. They propose, also, to attempt to tie up coastwise and transatlantic shipping, and to try to enlist the Sandy Hook pilots. The strike is also aimed at the Railroad Administration. If effective with regard to that organization, a serious menace to the coal supply of New York is in sight.

The speaker will not discuss the merits of the proposed strike or the conditions which make possible such a deplorable state of affairs, where a comparatively small body of men may actually endanger the physical existence and paralyze the economic life of a city of 3 000 000 people in order to satisfy their own particular desires. Such conditions give added emphasis to the idea that the development of New Jersey's highways to their highest efficiency and the construction of vehicular tunnels under the Hudson River are intimately connected with the future welfare of New York City.

The construction of one or more vehicular tunnels under the North River, if such tunnels are wisely designed to provide for future increase in vehicular traffic between New Jersey and New York, will go far toward removing the possibility of a future food shortage in New York City.

It is granted that freight and foodstuffs may be brought from Long Island, from New England, and from the West, *via* Buffalo and the Poughkeepsie Bridge, which in a measure would alleviate conditions arising from a future tie-up of harbor transportation; it is a fact, however, that by far the greater proportion of the food and daily necessities of New York City come through the freight-yards on the New Jersey shore, and that even a slight interruption in the continuous flow is serious, when it is considered, as has been asserted, that Manhattan Island has but 7 days' supply. It is also admitted that coal and other freight may be brought through the Hudson River Tunnel to the Pennsylvania Railroad Station at 34th Street and Seventh Avenue, but the passenger terminal lacks adequate facilities for freight handling, and cannot be considered an important factor as a freight-receiving and distributing point without serious interruption to the passenger traffic; therefore, development of means of vehicular transportation between New Jersey and Manhattan Island at an early date is imperative. Relative to this matter, it is interesting and gratifying to note that Governor Edge of New Jersey and Governor Smith of New York have signed bills which will enable preliminary steps toward actual construction to be undertaken.

Mr. Thompson.

It is well known that the present ferry system is incapable of handling the vehicular traffic expeditiously. Long lines of trucks now stand many hours awaiting their turn to cross the river from both sides, and the delay, being costly to all concerned, naturally affects most heavily the ultimate consumer. Statistics, compiled in connection with an investigation made by the Public Service Corporation of New Jersey, show that 6 130 945 vehicles of all classes use the North River Ferries between 130th Street and Cortlandt Street annually. These figures are based on an average week-day record, multiplied by 300 for week days, and a Sunday record multiplied by 65 for Sundays and holidays.

The normal increase in vehicular traffic is estimated to be between 3 and 5% per year. Considering this fact, and, further, that the ferries are now unable to handle the traffic, that the possibilities of further expansion of the ferry service are exceedingly limited, and that, even if the ferry terminals could be extended, the North River is now overcrowded with craft of every description, it is evident to men who have studied the matter seriously that at least one tunnel should be constructed as soon as possible.

It is estimated that 2 000 000 vehicles per year will use the tunnel. It is evident from these figures that the present congestion and long waits at the ferry entrances will be reduced very materially, if not obviated altogether. How long the proposed tunnel at Canal Street will serve properly the needs of cross-river traffic is a matter for conjecture. Mr. Edward R. Viets, of the Service Recorder Company of Cleveland, states that the number of motor trucks increased from 60 000 in 1912 to 600 000 in 1918. As the increase in the use of motor trucks follows rapidly the construction of good roads, and the greatest road construction period the country has ever known is just opening, it is reasonable to assume that the annual increase in the number of trucks will be as great during the next 6 as during the past 6 years. As a consequence of this, and the natural increase in volume of business within 30 miles of New York City, one may reasonably look for a recurrence of the present congestion by 1928, if not earlier; it would seem, therefore, that even before Tunnel No. 1 has been completed, consideration should be given to investigating a site for Tunnel No. 2. There is ample precedent for such action when one considers the East River. Considerable courage was required to start the construction of the Brooklyn Bridge. Not very many years after its completion came Bridge No. 2, then Bridge No. 3, and then Bridge No. 4. The capacity of these bridges is now sometimes severely taxed, notwithstanding that there are many ferries plying between Brooklyn and Manhattan Island. The population of Brooklyn is approximately 2 500 000. The population of New Jersey in the metropolitan district is approximately 1 500 000. Consider these facts, and, further, that the construction of Hudson Vehicular Tunnel No. 1 will facilitate and

stimulate business to such an extent as to increase rapidly the population of metropolitan New Jersey, that there will be created the same conditions which made necessary the construction of additional bridges over the East River. The completion of the present New Jersey Highway System, and its further extension as finances and conditions warrant, will but add to the burden converging on the Hudson River between 42d and Cortlandt Streets. Mr. Thompson.

It does not require a long stretch of the imagination to picture New Jersey's highway system as playing an important part in keeping New York City supplied with many necessities, especially when freight terminal congestion (which is inevitable at such an ill-equipped port) makes the handling and lightering of New York City and foreign freight difficult and tedious, more particularly in winter. Just a word as to the scope of this highway system as now laid out: The State Legislature of 1917 created a State Highway Commission of eight members, two of whom are members of this Society, and six of whom are well-known business men. There was also established a State Highway System, comprising 655 miles of the main traveled routes of the State, which serve well its industrial and agricultural regions. They are all tied together, and, in effect, radiate from or terminate on the New Jersey shore of the Hudson. Owing to its geographical position, New Jersey is the throat through which must pass all traffic from New York and points east to Philadelphia and points west and south. An unusually heavy burden of foreign traffic is thus imposed on the highways between the Hudson and the Delaware at Trenton. As an indication of its volume and weight, a traffic census taken early in 1918 showed a total daily weight of approximately 7 900 tons, including vehicles, freight, and passengers on a part of the Trenton-Newark highway between New Brunswick and Elizabeth. This is for one route only, and is an indication of the potential value of the highway system as a supplement to the railway system, especially considering the rapid increase in the number of motor trucks, which will follow the construction of pavements adequate to sustain them. Within one year there will be hard, smooth pavement on good alignment and easy grades, from the Hudson to the center of New Jersey's richest agricultural district. This will make possible direct trucking from farm to market or consumer. Even now, fruits and vegetables are trucked from points south of Trenton to the New York market daily. The completion of the tunnel will enable this produce to proceed uninterruptedly to New York, instead of waiting many hours for ferry passage.

In order to provide a highway for traffic to and from Hudson River points without passing through the congested Newark streets, the Counties of Essex and Hudson, with aid from the State, are now making investigations looking toward the construction of a highway bridge over Newark Bay between Bayonne and Elizabeth, by which

Mr. Thompson.

vehicular traffic may reach the Hudson County Boulevard, and thence go directly to the tunnel and ferries, shortening the motoring time between Elizabeth and Jersey City by at least 30 min. The State Highway Commission, in building a new road over the Palisades to Fort Lee Ferry in Bergen County, is doing much to relieve the congestion at the lower river ferries. The major portion of truck traffic from the Passaic and Hackensack Valley industries must now use the down-town ferries. The proposed new approach will provide a direct route from Paterson to 130th Street, New York City. The grades will not exceed 6.5%, and will be much lighter than on any present approach over the Palisades to the ferries. This will also enable heavily loaded trucks, bound to or from upper New York State, Connecticut, and Massachusetts, to avoid down-town New York City, when en route to or from New Jersey and Pennsylvania points.

The completion of State Highway Route No. 9 will open a direct smoothly paved road from New York to the Delaware River at Easton, Pa., whence there is a fine paved highway to South Bethlehem. As a peace time highway, this will have great commercial as well as scenic value. As a war time highway, its value would be inestimable, as it would connect the great Schwab plants directly with the Port of New York.

All the great industries of the Delaware River as far south as Wilmington, Del., will be connected directly with New York City by hard, smooth highways, by the autumn of 1920. It is said that all roads lead to New York. It is certain that New Jersey's highway system leads traffic to the Hudson River in an ever-increasing stream. The Highway Commission is constructing only the most durable pavements, which, in effect, will be city streets traversing the entire countryside, bringing the farms and producing centers ever in closer touch with the large cities. New Jersey contemplates spending approximately $6 000 000 during the summer of 1919 on State highways alone, in addition to which the counties will spend approximately $2 000 000 for new construction. As a sidelight on highway development as an adjunct or supplement to future railway development, the speaker quotes from a recent statement by C. A. Morse, M. Am. Soc. C. E., of the United States Railroad Administration:

"The perfection of the motor truck and tractor, together with the universal use of the automobile, has introduced a new element into the transportation problem that should be taken into consideration at this time, while studying the reorganization of the whole transportation question. Good roads are demanded for the use of the automobile, and a study should be made to see what additional expense would be necessary to construct them so that they would serve for motor truck and tractor. Where, heretofore, development of the country for 50 miles on each side of a trunk line of railroad has required the construction of light branch lines, it is a question to be seriously considered whether

this policy should be continued or whether good wagon roads should be constructed and the products of farms, and passenger travel, should not be handled by motor trucks and automobiles to the main line. Taken alone and considered as a unit, practically none of these small branch lines pays expenses, but as gatherers of freight and passengers to increase density of traffic on the main lines, they are sources of profit. As, however, the traffic gathered by them is turned over to the main line with a deficit attached, which has to be overcome during the main line movement before any profit is made, it would be a decided advantage if this traffic could be delivered to the trunk line by means of the motor truck, tractor, and automobile without this bill of expense attached." Mr. Thompson.

Noting the fact that thus one handling would be obviated, as it is now necessary to truck farm products to the short-line branch, and then transfer them to the main line, Mr. Morse continued:

"Investigation of this subject may show the desirability, as good roads are completed, of taking up many branch line railroads and utilizing the abandoned roadbed for improved motor roads, thus decreasing the expense of maintenance and operation of our railroads and giving in its place a well located motor road. Such a change would call for increased facilities at stations along the main line for passengers and for hauling freight, including storage, trackage, etc., but it would mean the concentration of supervision and labor, permitting better housing and living conditions for employees. Motor-driven conveyances have gradually been changing conditions of railroad transportation for the past ten years, and now that the general study of transportation facilities is up for discussion, it should be taken into careful consideration and due weight given to its influence on the economies of the situation."

From these quotations it will be seen that railroad managers are seriously considering the question of leaving to the highways the matter of caring for short-haul freight. Such a system would obviate the construction of short feeder lines, costly rehandling of freight, and result in more rapid delivery.

The motor vehicle laws of New Jersey are now very well defined. The maximum load allowed is 30 000 lb., assuming that approximately 75% of that load is carried on the rear axle. The speaker thinks that is higher than is allowed or contemplated by most States. The tire equipment is also regulated in accordance with the weight of the machine. The speaker thinks that the maximum now allowed is 800 lb. per in. of width of tire.

The regulations of the Motor Vehicle Department also restrict the height of trucks to 12 ft. 6 in., and the maximum width to 8 ft. The traffic regulations are also about to be enforced more rigidly, in order to provide greater safety on the highways. The Motor Vehicle Commissioner has fifteen or twenty inspectors who are patrolling the roads

Mr. Thompson.

constantly, in order to protect the careful driver against infractions of the regulations.

The question of determining a standard or a set of standards for the regulation of motor trucks to restrict their weight, size, and speed, is now under way, a committee having been appointed by the National Highway Traffic Association. This committee is composed of five State Highway Engineers and four representatives of the motor truck manufacturers and users.

The matter came up for report at the Annual Meeting of the American Road Builders' Association, held in New York City, in February, 1919, when the committee made a Progress Report to the effect that the weights of motor vehicles should be restricted arbitrarily to 28 000 lb. gross weight of truck and load.

That met with considerable opposition, because of the fact that at present the statutory requirements in a number of States are much lower, running down to 24 000 and 20 000 lb. gross load. The question is debatable, as the decision depends entirely on arbitrary conclusions, because there is no definite data to determine how much more damage may be done by a 28 000 or a 30 000-lb. load than by a 5 000 or 10 000-lb. load; but it is hoped that during 1919 a common standard will be accepted, at least, for the North Atlantic and the Eastern States of the country, where the truck traffic is now the heaviest.

Probably the matter will be settled eventually on a basis of about 25 000 lb. That was demanded by the highway builders and highway authorities, and an attempt was made to show to the motor vehicle owners and motor vehicle manufacturers that unless there was an arbitrary decision, the agitation in communities having to pay for those roads would be such as to compel them to adopt a lower standard. For example, the Legislature of Pennsylvania had a bill before it restricting the weight to 24 000 lb., which would have compelled trucks which could be used in New Jersey up to 30 000 lb. to stop at the Delaware River; and, of course, that would interfere with the free movement of interstate truck traffic.

New Jersey's law is in operation now, and an effort is to be made—a very effective effort—to see that trucks do not exceed that load, because the inspectors will compel the weighing of occasional trucks which seem to be too heavy. If they are over weight, the owners will be fined for disobeying the law.

Mr. Breed.

H. Eltinge Breed,* M. Am. Soc. C. E.—The present is the point of departure. Before discussing concrete details of future development, an attempt will be made to state briefly the present conditions. The war has given a tremendous impetus to all the sciences. The economic situation stresses to-day as never before the necessity for ampler, fairer methods of production and distribution. Engineers have a mass

* New York City.

of definite scientific data behind them, and the needs of a world before them. Mr. Breed.

Highway work has now reached the adolescent stage. Its infancy lasted more than 100 years, about 90 of which were spent in one long nap. Under Jefferson's administration the National Government and a number of the States started a road-building programme. The Government built a few hundred miles of roads and the States a similar number. Shortly after starting this programme, all road building on the part of the National Government and the States ceased. The work was then taken up by many turnpike companies, which built a number of roads in the Eastern States, operating them under the toll system. After the establishment of the toll road came the "nap". Road construction was handled principally by the township and county officials, with no development in types of construction or methods of maintenance, outside of large cities. This condition continued until the late Nineties. Then came the bicycle craze and active propaganda on the part of the League of American Wheelmen for better roads. Before this died out, the automobile had come to stay. As the number of cars increased, the great concern was the dust nuisance. Oil was applied and bituminous binders were used, but, almost before they were on, the old earth and light gravel roads beneath them had begun to disintegrate. The road builder's problem was growing, and he tried to solve it by constructing a better surfacing for the macadam types. That did not last long, as the increasing weight and speed of vehicles, and a few spring thaws and freezes, broke it up.

This also affected the taxpayer's money, for, by this time, the various States had taken up the matter of road building, with subsidiary county and township systems. In New York State the Legislature of 1898 passed two Acts, the Fuller-Plank Act, providing that the State would duplicate for each town whatever amount it raised for highway purposes; and the Higbie-Armstrong Act, appropriating $50 000 for roads to be improved—50% by the State, 35% by the county they were in, and 15% by the townships. Each year after that the annual appropriation for highway work grew, until, in 1904, it was $1 108 000. Yet there were only about 300 miles of even average good roads.

Some saw that it was time to call a halt, and cried, "This road building is no job for political henchmen; it is no game for amateurs. We need trained men. Give us university courses for Highway Engineers. Give us laboratories for experimental work. Above all, let us start at the beginning, plan our routes, abandon cow-trails, and build our roads from the bottom up."

That happened about 1903. Since then road engineers have been trying to keep up with traffic, like a child chasing a rubber ball down hill. They had almost caught it, when, suddenly, came the motor truck. Loads of 15 or 20, and even 30, tons were carried on roads designed

Mr. Breed. to endure 5 or 6 tons, and the roads broke down. Before this policy could be re-shaped, war was declared, and motor trucks were being built in great numbers in order to supplement the work of the railroads. Maintenance on the route along the east bank of the Hudson between New York City and Albany for 1917 cost $215 200.74, in comparison with an expenditure of $115 816.56 in 1915, showing an increase of almost $100 000, due to heavier motor truck traffic; but the roads were kept open for traffic, and there was no cessation of men and supplies between Buffalo and New York City. That, and the fact that to-day there are in New York State 7 200 miles of improved roads and 15 completed routes, north, south, east, and west, through the State, is a measure of the success attained. This, however, has been extravagant beyond measure, in money and energy, and has been smirched with failure. Politics have not been kept out of highway work. Some through routes lack important links, because, instead, spurs have been built to the homes of influential politicians. The work has not been kept out of the hands of amateurs—the roads are full of them, and will be until highway work pays bigger salaries and demands higher skill. Engineers have not done enough laboratory work. Instead, they have been prone lazily to trust the statements of the material interests, and this costly experience has not always been uncolored by commercial fiction. Only now are they beginning to understand what the term "foundation" really means.

The speaker has said that highway work has now reached adolescence: First, because a clear perception of the problem ought to mean that engineers have outgrown the errors of infancy.

Second, because public sentiment is solidly behind the idea. It has been created in the only real way, by the discernment of the people themselves that they must have good roads in order to be fully a part of the world in which they live, and has been formulated by the well-conducted campaign of good roads and automobile organizations, motor truck, and material interests.

Third, because the money has been provided, and more is coming. The Government has appropriated $200 000 000 as Federal aid to roads, and is planning to provide another $400 000 000 for National systems. New York State is on its second $50 000 000 bond issue. Illinois and Pennsylvania have just raised $50 000 000 and $60 000 000, respectively, and other States are rapidly falling into line. If these moneys are expended economically, no shortage of funds need be feared.

Fourth, because the people have reached the psychological condition which characterizes adolescence, namely, self-scrutiny and analysis.

What is meant will be explained in terms of types of pavement. On January 1st, 1918, there were, in New York State, 6 912 miles of pavement, of which 5 820 miles were non-permanent types and 1 092 miles were permanent; of the latter all but 129 miles were of brick or concrete.

The non-permanent types are water-bound macadam and bituminous macadam by the penetration method. Mr. Breed.

Water-bound Macadam.—This pavement is best adapted to the needs of light traffic, but has no place on any through route. It costs to-day about $23 000 per mile for a 16 to 26-ft. pavement, 9 in. thick (that is, a 3-in. top on a 6-in. sub-base). Low construction cost has been the greatest advantage. Its greatest disadvantage now is its high cost of maintenance, which, in 1917, amounted to $970 per mile. The average life of a pavement of this type is 7 years for a traffic of less than 1 000 tons per day. Many water-bound roads have worn out in from 1 to 3 years when required to carry a heavier tonnage.

Bituminous Macadam.—Penetration Method.—This type is also adapted to light traffic, generally, and is advantageous in many localities where the only stone available is non-binding. Among its disadvantages is its tendency to creep under traffic. It is slippery when wet; has a fairly high cost of maintenance—$412 per mile during 1917—and a construction cost of $28 500 per mile for a 16 to 26-ft. pavement, 9 in. thick (that is, a 3-in. top course and a 6-in. sub-base bottom course). The average life of this type is 2 years more than that of water-bound macadam under similar traffic conditions.

The determination given for the average life of the foregoing types was made in 1917. The much heavier traffic tonnage of to-day would shorten it considerably.

The following are classed as permanent types:

Concrete.—This pavement has the disadvantage of a glare. It has the advantage of low first cost—$31 200 per mile to-day for a 16 to 26-ft. pavement, with an average thickness of 6 in. Its maintenance in 1917 was $112 per mile. Its average life, thus far, can only be estimated, but, judging from the condition of pavements of this type, from 5 to 9 years old, with which the speaker is familiar, it is, say, at least 15 years.

Brick Pavement.—Brick pavements of the sand-cushion type have not been entirely satisfactory for country roads in this climate. One disadvantage is the shifting of the sand cushion, with the consequent settlement, breaking, and spalling of the bricks. The speaker cannot give present-day costs, as the State has ceased to build brick pavements, except as semi-monolithic and monolithic structures. The cost of maintenance was $222 per mile in 1917. The average life, of course, can only be estimated, but, from present indications, it might be placed at not more than 12 years.

Brick Pavements.—Semi-Monolithic Type.—This type differs from the original brick pavement, in that there is no sand cushion, the bricks being laid in a cement-sand bed and treated with cement grout so that the bed forms an integral part of the pavement. This type has the advantage of developing beam strength, as in concrete pave-

Mr. Breed. ment, and it eliminates the shifting of and depressions in the brick. Its cost under present conditions would be about $50 600 per mile for a 3-in. brick on a 4-in. concrete foundation. The speaker has no maintenance figures for this type, as the State has been building it for only a little more than a year.

Brick Pavements.—Monolithic Type.—This type differs from the semi-monolithic in that the bricks are set in a green concrete and treated with a cement grout so that it forms an integral part of the pavement. This type develops the full beam strength, and has even greater advantages than the semi-monolithic type, so far as strength is concerned. It is more difficult, however, to get as smooth a surface as can be obtained with the semi-monolithic type.

Asphaltic Concrete.—In order to insure a nearly smooth pavement, a smooth concrete foundation should be provided. These pavements have the disadvantage of pushing under traffic from future consolidation, if the aggregate is not properly sized and proportioned, or if it contains too much bituminous material. However, if too little of the latter is used, the pavement becomes dry and soon disintegrates. The ideal mix is one in which the motor oil dropped from cars renews the volatile oils lost. These pavements cost about $40 000 per mile (depending on the type), with a 2-in. top and a 5-in. concrete foundation. Their maintenance cost in 1917 was $245 per mile.

There are many patent pavements which come under the classification of permanent types and are good.

It is apparent that engineers know, with a fair degree of precision, what service such roads as these, built as they are to-day, will give under present traffic. With this knowledge, which was entirely lacking even 10 years ago, the speaker thinks they are prepared to emerge into real maturity. However, there is one proviso, namely, that no load on the roads shall exceed a given weight. In this restriction they should be guided, for 5 years at least, by the experience of the past. The slight chance that it may be a limitation is more than offset by the absolute assurance that it will be a safeguard to the roads.

The motor truck marks a distinct transition in transportation facilities. It has made the highway as available for freight haulage as the waterway and the railway. The service of the motor truck in the freight crisis of the past year established forever the importance of the highway as an integral part of the transportation system of the United States. Thus far, however, it has been fatal to the very factor on which its usefulness depends—it is the greatest destructive force the highway has ever encountered.

The latest traffic reports, on fifteen highways in various parts of New York State, show an increase in truck tonnage varying biennially from 50 to 400%, and, in 1918, an average increase of 150% over the

tonnage of 1916. To try to build highways to keep pace with a continuous advance of that kind would bankrupt the public. Mr. Breed.

The wiser motor truck manufacturers have themselves adopted the broad policy of selling transportation service, rather than the exclusive one of selling motor trucks. These men, who have had every opportunity to study the economics of the problem, have agreed on the desirability of the 5-ton truck as a standard unit. This should now be established by law, and the highways should be designed to meet it. It will mean a total maximum load of about 12½ tons, of which approximately 9 tons should be applied to the rear axle. There should also be general regulations for other destructive factors, such as inadequate tires and springs.

Engineers can build roads economically for the 5-ton truck, but cannot afford to have them disrupted by a few excessively large trucks. Nor should they be designed extravagantly to meet the requirements of a few individuals. The maximum loading must be restricted to the economic standard suggested by the manufacturers and desired by the majority of truck users. Then will the highway be able to give to the motor car and to the community its most efficient service.

Probably in every other line of engineering there is some such restriction, based on a fundamental and enduring economic law of relative values. Beyond that, however, engineers can plan for the future with imagination unfettered by any narrow limits of probability.

The speaker believes that the country, during the next 5 years, will be criss-crossed by myriad lines of communication: Waterway, railway, highway—the trinity of transportation, with great cities along the first two—centers of world-wide distribution, and feeding them, the highways with motor trucks incessantly going back and forth 10, 15, 20 miles, to centers of production, flourishing agricultural and industrial towns. Manufacturing will be more and more forced out of the big cities, through pressure of space, and into the surrounding country, where the much cheaper rentals will be more than an offset for the slightly greater carrying charges. Highway and motor truck make this feasible, and independent of railways.

Farming must be encouraged if the people are to live, and the speaker sees the highways as a great means of encouragement to the production of farm wealth. They will help banish from farm life its deadly isolation; they will make possible community towns, from which outlying farms may be worked. They will help attract men back to the land, and then, with the motor truck, help them market their products with less intervention by profit-taking middlemen.

The trunk roads from the great cities to the smaller towns, the subsidiaries leading from them through the more rural regions—all these will be united in great systems that will provide through and uninterrupted passage from any place in the country to every other place.

Mr. Breed. Guiding this work of co-ordination, there will be, in Washington, a National Highway Commission, second in importance only to the Cabinet Departments. Beyond the proposed Lincoln Highway, that Commission will have under construction, or completed, before 1925 at least four other transcontinental highways, planned solely from the standpoint of highway economy, without reference to existing cities and towns. Given the roads, the population will settle along them, just as it has along the railroads. Throughout history civilization has always followed the lines of communication. At first, Nature determined these lines; now they are determined by Man.

Meanwhile, each State has been perfecting its own system within its borders, building through routes between its important points, with whatever spurs are necessary to connect with the National system and with similar routes in adjacent States; and the counties have helped, with feeders to these routes from every remote village and hamlet, until all are sharing to the fullest extent in the surge of national life.

With such development for the highways, with the suggested improvement of the waterways, and the hoped for resurrection of the railways, the transportation facilities of this country 5 years hence ought to be more than adequate to meet the demands on them.

So much for the vision—akin to what every one probably holds for his own special line—now for the toil to make it real.

First of all, in every line of engineering, one must work for greater permanence in result. Europe has proclaimed that lesson, but this country has not heeded it. With the vigorous impatience of youth, stability has been sacrificed to speed, size has been acclaimed rather than soundness.

The public must be roused to demand that engineers build deep and well. They must learn to spurn the shoddy, beloved of their politicians. Last week a delegation from an up-State county came importunate to the Commissioner's office in Albany. "Gentlemen," he said, "I want you to have these roads, but I have exactly $300 000 to spend in your county. Would you rather have 10 miles of good road that will last you 15 years without ever having to be closed for repairs; or 20 miles of poor road that may last for 3 years and will have to be closed often?" With one accord they cried, "Give us the 20 miles."

At the speaker's remonstrance, one member of the delegation said, pityingly: "The counties want the roads. The cheaper they are, the more we get for our money in them. The State has to do the maintaining, and most of the State's taxes are paid by New York City. What is our worry about permanence?"

It is a worry, however, that engineers cannot dodge. The National routes should have foundations at least comparable with the best railroad practice, and be surfaced with one of the permanent types—the choice depending on the materials most available in a given locality.

The State routes should have the same surfacing and strength of foundation. Within a 30-mile radius of the large cities, the foundations must be still further deepened and the pavement widened to sustain the heavier traffic. The county roads should have a semi-durable surfacing, according to the need shown by a traffic census, and a proportionately good foundation, except within 20 miles of the larger towns, where they should again be of permanent type. Mr. Breed.

So much for the basis of permanence. There are many other factors in a roadway, however, besides foundation and surfacing. They come under the heading of "details", through closer attention to which, engineers, in every line, may find opportunity for much improvement. A few may be mentioned which are of fundamental importance in highway work, and yet have hitherto received rather cavalier treatment.

Width of Roads.—The least width for National roads and most of the State routes will be 20 ft. for the pavement proper, with an additional 5 ft. or more for the shoulder; that is, where traffic density requires only two tracks; for each additional line of traffic, there should be an additional 9 ft. of pavement. On a few State roads, where traffic is lighter, and most of the county roads, there will be a 10-ft. pavement of hard surfacing and beside it 10 ft. more of earth road which may be improved when necessary to make the full two-line traffic pavement.

Alignment and Grades.—Although the straight line shortens the distance, curves are often necessary in order to conform to the broken topography of the country. This matter of alignment and grade will receive more careful study in relation to economizing the fuel used in the propulsion of motor vehicles. All curves should be reduced so as to afford a sight distance of 600 ft. Where that is impossible, the curves may be split by a parking place or posts in the center so as to divide the traffic. Buildings will be removed from dangerous corners, and banks graded back; and, wherever possible, curves will be widened so as to secure greater safety for passing vehicles.

Drainage.—No matter how well a road has been drained, its use generally shows that more drainage is necessary. Ditches must be widened as foundations are deepened. Only by keeping the subsoil dry can it act as part of the foundation, cheapen the cost of the pavement, and insure coherence of crust during wet subsoil conditions. The speaker foresees the return of the deep ditch far enough away from the roadway proper not to imperil the traveling public, and providing run-offs and lateral systems to drain away the sub-surface water.

Marking of Routes.—Is it Utopian to hope to have, at every turn, adequate sign-posts which one can read when going at a speed of 10 miles an hour? There is no doubt that there will be general acceptance of the banding of poles in different color schemes to indicate main

Mr. Breed. traveled routes, so helpful has this procedure proved in New York and the New England States.

Esthetic Features.—An ugly sign-post is a public offense, so is the billboard and the unnecessary danger sign. They will not be tolerated. On the other hand, the furtherance of the Arbor Day and patriotic memorial idea will enhance the beauty of the roadways with shade trees and shrubs, and parking spaces must be provided at definite intervals, and also the sorely needed rest and comfort stations. A greater length of culvert and the general use of the straight line in its design will obviate the need of unsightly guard-rails around its ends. The use of boulders and rustic guard-rails will be an improvement over the ugly painted wooded affairs now used, and they will also be cheaper.

Letting of Contracts.—Because of the greater quantity of work to be done and the scanty number of contractors to do it, there is need for encouraging large construction organizations to enter the highway field. This will mean ultimate economy. A large number of contracts will be let in lengths of from 15 to 30 miles, justifying the use of special equipment and special labor-saving devices, such as the short contracts cannot warrant, because of the disproportionate overhead charge. The most adequate equipment is necessary to-day to supplant the rapidly diminishing quantity of hand labor.

Maintenance of Traffic.—Many of the old roads will have to be reconstructed in order that they may take their place in the new systems. Experience in New York State has shown that where a semi-durable pavement has broken up because the weakness of a soft and yielding subsoil has been accentuated by heavy traffic during the spring thaws, it has been cheaper in most cases to fill the broken gaps with a rigid type of pavement—of concrete or with a concrete base—rather than go to the expense of deeper excavation for extra drainage and for extra thickness of sub-base foundation under the old type. These gaps then become a permanent start toward the reconstruction of the whole road.

In building pavements of the widths proposed, it will be possible to build one longitudinal half at a time, and thus provide for a long-suffering public that has been discommoded with insufficient detours.

Thus, are suggested some of the possible developments of the next 5 years. Their practicability to each engineer depends on his experience, viewpoint, and temperament. They may not prove practical at all. Then they must be modified or changed. That is of less concern. What greatly matters is, to be free from too close dependence on the past, to see the future with larger vision, to apply permanence as the one standard of success, and to give to details the devoted attention of artists and scientists. These traits, sought in common, will

strengthen the alliance among us all, and, to these traits, the speaker believes, will be due the greatest good that may come from the highways of the future. Mr. Breed.

In reply to the inquiry as to the comparative merits of tar and asphalt in surfacing, the speaker finds it difficult to generalize. There are certain kinds of stone in New York State, such as gneiss, from which the best results have been obtained with tar. There are other rocks, like trap, from which the best results are secured with asphalt.

This can probably be explained better by saying that tar has a tendency to go down through the rock. The gneiss rocks are more friable, and hold the tar up better, but with them asphalt has a tendency to work up or bleed—this is a condition which probably all know—if it has not thoroughly penetrated. With the harder rocks, it is found that asphalt gives the best surface, because it penetrates the interstices.

The tendency of tar to go down often makes it necessary to surface treat the road with cold tar and fine stone, sand, or gravel. If this is not done, the surface will soon become toothy and pitted.

EUGENE W. STERN,* M. AM. SOC. C. E.—The preceding speakers have not exaggerated in the least the importance of the highway problem in the United States. It is a very live subject, and one that demands the fullest discussion and propaganda for enlightenment. Mr. Stern.

It is hoped that this Society will take a much more active part in the propagation of information on this matter than it has in the past, and that it will encourage, in every way, meetings for the discussion of all matters pertaining thereto.

In America, with few exceptions, road problems have not been attacked with proper vision and boldness. Macadam roads have been built, when hard paved surfaces should have been laid. Paved surfaces have been too narrow, curves have not been easy enough, grades have been too steep. Drainage has been neglected, shoulders have been neglected, and, most important of all, proper upkeep has been neglected.

There have not been generally adopted proper regulations limiting the size, wheel loads, etc., of vehicles. Organization and administration of highway bureaus have been niggardly, poor, inefficient, and utterly behind the times. There has been lack of comprehensive plans, and of co-ordination in linking up highway routes; and, lastly, there has been too much politics in the entire highway problem. The foregoing deficiencies are causing the loss of many millions of dollars annually to the taxpayers, and they are all easily preventable.

Some experiences on road work in France, where the writer served for more than a year with the A. E. F., may be of interest.

* New York City.

Mr. Stern. The administration and organization of the French road bureau is under civil service rules, and is efficient and excellent to the highest degree. Politics does not enter into it. All road officers are trained engineers, graduates of the Ecole Polytechnique and of the Ponts et Chaussées, and are retired at the age of 62 on a pension.

The organization for maintenance and repair is very thorough, and is admittedly the best in existence. Men called *cantonniers* are stationed along important roads at distances of from 3 to 6 km., living in little houses along the roads. They are at work continually, and never allow the roads to get in poor condition. Even if there is wear, the repairs are so skillful that the surface wears down smoothly.

British road officers have told the speaker that the roads in France are maintained at much less expense than in Great Britain, owing to the better organization.

Even in war times, it was surprising to see how well their organization functioned, though, of course, it was impossible to do the work as well as in peace times, as men and materials were often not available. Women and old men maintained the roads, back of the zone of operation of the armies.

French roads are thoroughly drained, every precaution being taken to carry away the water, and keep the sub-base dry. All roads have side ditches, and the National roads likewise have gutters, draining into the side ditches. The ditches and gutters are kept scrupulously clean, the grass being close-cropped, and the bottoms kept smooth. The water in the ditches is always kept flowing.

Shoulders are grassed, and trees are planted on both sides, and serve a double purpose, absorbing the excess moisture from the shoulders, thus tending to prevent frost heave, and, of course, the trees add very much to the beauty of the road.

Nearly all French roads are water-bound macadam, the important roads having a Telford base, only a small percentage being paved with granite, or treated with tar. They are wonderfully well built, the greatest care being taken in every detail of their construction. The macadam surface is made with one-size stone, $2\frac{1}{4}$ to $2\frac{1}{2}$-in. trap or granite, most thoroughly consolidated with a heavy roller, usually weighing 17 tons, the stone being kept thoroughly wetted during the process of rolling. The roller passes over the surface from 150 to 175 times, and, when the job is done, the surface is so thoroughly consolidated, and the stones so interlocked, that it looks like concrete. Regardless of the great excellence which the French have attained in the building of macadam roads, however—probably the very best of their kind in existence—the war proved beyond any question that they are not suited for motor vehicle traffic. Shortly after the motor trucks of the U. S. Army commenced to use these roads extensively, they wore out rapidly, becoming full of holes and ruts. It required a

great deal of labor and huge quantities of stone to keep them at all serviceable. Mr. Stern.

If there had been hard paved surfaces on the main highways in France during the war, millions of dollars would have been saved, and a great burden on the railways and canals would have been avoided, as about 40 000 tons of road stone had to be transported daily for the use of the armies on the Western front alone.

In France, before the war, few motor trucks were used, and these mostly in the cities, or for a short distance outside. When war came, this situation changed almost over night. Every available motor vehicle was requisitioned, and the traffic developed by leaps and bounds. The motor truck saved France.

The French road engineers, although capable men, and not lacking in vision, had an entirely new problem to meet, but they could not improve the road surfaces in war time. It was too late. They were compelled to use macadam for all repair and new construction, and now they are frank in admitting that it will not meet the necessities of the future. The upkeep costs far too much.

In the United States, the great question today is—"How can we best meet the road problem?" The speaker believes that it can be solved as readily and efficiently as other engineering problems, if it is attacked in the right way.

First.—Road building must be taken out of politics, and put in the hands of capable men. Highway commissioners should be able engineers, of thorough training in road work, who possess vision and force of character, combined with administrative and executive ability of a high order. It has been only too common for highway commissioners to be politicians, lawyers, or of any class not meeting the above requirements.

Second.—The remuneration and conditions surrounding the service should be made sufficiently attractive to induce professional engineers of a high grade to follow this calling. An engineer in responsible charge of road planning and construction should have certain qualifications. Besides technical knowledge, he must have vision—imagination for the future—balanced by good judgment. If he does not have this, and the courage of his convictions, he is in a position to waste huge sums of the taxpayers' money. His decision alone, as to what type of wearing surface should be used, may mean great saving or loss to the community.

There is an example of this in New York, and in many other States, where macadam roads have been built in recent years, when every indication pointed to the lesson that surfaces of this type would not stand the growing motor vehicle traffic. The result is that there are many thousands of miles of macadam roads costing annually for upkeep alone more than $1 000 a mile, whereas, had a road of proper

Mr. Stern. type been built, the annual gross cost to the taxpayers would have been almost cut in half. By annual gross cost, the speaker, of course, means interest on the cost of construction, plus amortization, plus annual upkeep.

The speaker has only touched on the more important considerations in the solution of the road problem, as he believes these to be fundamental. Given proper organization and leadership, the rest will follow.

Mr. Tomkins. CALVIN TOMKINS,* ASSOC. AM. SOC. C. E.—The speaker calls attention to two or three points which have impressed him in the discussion of this interesting subject. On the New York side the highway system is completed, and also beyond the meadows district in New Jersey, where there is an excellent system radiating from the Passaic River; but, in the intermediate meadows district, there is lacking any kind of road system, or any plan, so far as the speaker knows. There are three principal roads crossing the meadows, and no adequate transverse roads running north and south between them, as there should be. Before industrial development proceeds to any great extent in this territory, a general plan of road construction should be adopted, in anticipation of the needs; otherwise, it will be necessary, in the future, to go to very great expense in order to provide a suitable system. The roads should converge on the meadows as much as possible at a few crossings on the Hackensack and Passaic Rivers, so as not to interfere with the river traffic any more than can be helped. This interference of water and land traffic has already become a serious problem.

The speaker doubts the advisability of building a bridge over Newark Bay, as the latter is second in importance only to New York Bay, as a part of the Harbor of New York. In its present condition it is a shallow body of water, and of little use; but it can be dredged, and at far less expense than that incurred at Bremerhaven, Glasgow, or Hamburg, and made a most important adjunct to the harbor. Bridges across it will interfere seriously with its navigability.

The speaker believes the time has come when a system of tunnels under the Hudson should be planned in advance. These first vehicular tunnels are only the precursors of many which will be built, and the lay-out of the system should be such as to prevent interference. The conditions in the bed of the river are known, and it is of the greatest importance to leave those conditions so that many tunnels can be constructed. The speaker has been informed that there is in existence a plan for an Erie Railroad river tunnel, running diagonally across the river, which, if built, would interfere seriously with future east and west tunnels.

* New York City.

Mr. Thompson's remark relative to the utilization of small unprofitable railroad feeder lines for highway uses is important. Highways have demonstrated their efficiency as feeders for the railroads. The small railroad lines are not economical, and, just as trolley construction has been stopped by the building of good roads, so feeder railroad lines will be stopped, and many of them can be used advantageously as highways. Mr. Tomkins.

However important vehicular tunnels may be for communication between New York City and New Jersey, one should not lose sight of the fact that the fundamental needs are railroad tunnels. They are needed as soon as they can be built, and a strike would be a fortunate rather than an unfortunate occurrence for New York City if it helped to demonstrate the absolute necessity of establishing physical all-rail connections with the great continent to the west.

The newspapers state that only 20% of the population of Petrograd remains. The rest have been starved out because of the desperate consequences of the interruption of railroad traffic. The same causes may eventually produce like results in New York City unless continuous railroad service can be maintained with the continent. At present, there is only the intermittent railroad lighterage car-float services across the harbor, and the traffic will be interfered with continually by labor disturbances, because the harbor transfer is the weak link in the system, and will be the point of attack every time a railroad dispute arises.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—The discussion on American Highways is full of interest and encouragement. The encouragement is obvious, and was to be expected, because many conditions are somewhat novel and, therefore, the pendulum is liable to pass the ordinary limit. When this occurs, it is natural to accelerate further instead of imposing restraint. It is more common and popular in any event. Mr. Dunham.

A few lessons from the past should be cited and heeded. When it was shown conclusively that electric traction offered certain advantages in certain localities, scant regard was given to the fundamental rights of existing lines of steam transportation, or to the true interests of the general public. In that period of enthusiasm, it was held in some States that to permit an electric line to carry freight would be to treat unfairly a long established steam road in the same vicinity, and such permission was denied, not in a particular case, but in all cases. Thus, many electric lines that might have been valuable feeders to the steam roads and of great value in their districts were discouraged and not built. At the same time, steam roads between cities that had been almost created by those roads, were linked up, with little or no objec-

* New York City.

Mr. Dunham.

tion, by electric lines that are now in need of more passengers, higher fares, or new investors. The errors in the two methods thus presented, opposed alike to good morals and good legislation, have helped to impoverish the public, weaken all service, and introduce chronic troubles.

These are facts that should be examined and studied well before entering on an almost identical course following another change or advantage in the application of mechanical power. Had a different course been followed, and electric lines located and given rights where the best interests of all concerned demanded, the two systems might be working in harmony in many places throughout the country where serious difficulties now exist; and, further, a close approach to that more satisfactory condition could have been reached, if the demands of promoters had been limited by competent railway boards or public service commissions.

After this broad object lesson, and in the midst of further activities, the United States Government will not set aside funds to be used in the improvement of a highway unless that highway is now "a post road", while it is notorious that post roads, especially in New England, were located over summits nearly as high as could be found. At the present time, and for this reason, improvements are made following bad locations and the most difficult grades. First, there should be better co-operation between Federal, State, county, and municipal authorities. Labor interests should be included, but it may be questioned whether wrong labor activities, that is, wrongly or badly planned, should lead to extended new construction, even if such new departures would require the employment of many workers. Recently, we have seen that mere numbers are not an effective safeguard against injury to all classes. Parts of the United States and provinces in Canada and Russia show this. Where a weak link exists because of active corrosion, it is better to safeguard the weak place than to add elsewhere to surfaces that must be exposed to the same acid influence. Engineering would soon be a synonym for impoverishment if wrong deeds always insured new and extended construction. One may question and doubt whether engineers should encourage larger factors of safety, because of political unrest among American workmen.

In electric road construction, it was generally intended that the cost of right of way, construction, and equipment should be covered by bond issues. The stock became valuable from earnings in excess of interest and operation charges. Profits increased as maintenance, equipment, and depreciation were neglected. Often the roads were left in the possession of the bondholders to be operated or abandoned. All this was individual or business enterprise, where bad judgment, want of foresight, or too great enthusiasm brought discomfort.

Mr. Dunham.

Now, however, in the new adventure, where the rights of way and the roadbed are owned by the public and, of necessity, kept in some kind of condition by that public, complications are increased immensely. The Ford automobile and the heavy truck have practically equal rights. The light rig and the locomotive are on the same roadbed without rails, and both look to the public for support and safety. A requirement of one is that no special diversion of rainfall shall occur at intervals on high grades. An evil of the other is a groove in the roadbed that becomes a destructive river in every hard storm. The public makes repairs. Where the ground is soft, the heavy truck depresses the roadbed. The public brings the surface up to grade. Deep freezing and thawing make radical changes in the surface. The public attends to that. In winter, drifting snows bury fences and fill up the cuts. The public opens the way without the aid of a snow plow.

It seems worthy of remark that in a discussion on highways in a State the latitude of which is 42°, and where the snowfall is exceptional, not a word was said bearing on snow blockades or the expense of keeping a bridle path open in weather that closes up the Hudson River. The only reason for this must have been the confidence engineers share in the assistance to be derived from the public. We know what has been done under the stress of war, when long trains of trucks were brought through deep drifts in the mountains. We seem to think a similar service can be maintained at public expense.

In reaching out, however, into districts not occupied by railroads, it should be remembered that throughout a large part of the United States winter service must be abandoned above the snow line, and for no small part of the year the equipment is idle. To that extent this method of transportation suffers in comparison with railroads.

On steam roads in this country, freight is now moved at an average cost of not far from 2 cts. per ton-mile, and it is on such figures that determinations of profit and loss are made. Back of the actual rate, whatever it may be, there are years of operation with accounts that disclose almost every item of expense. How much is accurately known of the cost of hauling by motor truck per ton-mile over roadways provided and kept in repair by the public? Repairs to the trucks at this date are usually made by the owners.

In Massachusetts, steam road and highway automobile accidents are not returned under exactly similar headings. The Commission gives the number of passengers killed and a detailed account of accidents to employees on steam roads. It is not difficult, however, to determine how many employees are killed by trains in motion, and, in this way, to make fairly accurate comparison with the record of "Occupants of Automobiles."

Mr. Dunham. Thus, in 1918, in that State, 33 persons (13 passengers and 20 employees) were killed on steam roads. Of occupants of automobiles 136 were killed. The totals for the year may be thought to be more surprising: Steam roads, killed 247 (including 98 trespassers, not on grade crossings), injured, 137; killed by automobiles, including motorcycles and motor trucks, 499, injured, 8 598. These figures show a phase in transportation which cannot be disregarded in highway discussion and should be remembered when it is sought to change a steam road branch to a highway.

These losses in life, time, crippled limbs, and cost of recovery may not be classified in any particular schedule, but they are very real and are losses on the community balance sheet in any final auditing, which are and always will be on the wrong side.

Mr. Woolley. WILLIAM E. WOOLLEY,* ASSOC. M. AM. SOC. C. E. (by letter).—This discussion on highways is very interesting, and several of Mr. Breed's remarks might apply very aptly to English conditions. Many English highways which formerly were suitable for ordinary traffic, have proved to be totally inadequate when subjected to modern traffic conditions, with its heavy motor lorries, etc., owing to their non-rigid foundations and to the unsuitable material used for their surfaces. This has been most marked during the war, when fleets of heavy military motor lorries were in constant use, and at a time, too, when there was little available material and labor for repairs and a road-stone control besides.

Often in the past, the cheapest stone in the district was made use of because of its cheapness. This, in many cases, did not entail the use of durable and tough qualities of metaling. There has also been a marked absence of foundations suitable for carrying modern weighty vehicles.

In order to stand modern traffic conditions in towns, care, of course, must be taken that a thoroughly firm and solid foundation of sufficient thickness is provided, a suitable material being concrete which is being much advocated in England at the present time. The surface should be of durable material suitable for the particular kind of traffic to which the road is subjected. The road should be constructed on the most permanent and soundest principles. It has been established that ordinary macadam roads are not suitable for withstanding the heavy modern traffic in big towns. Weak, non-rigid foundations are often denoted by sinkage in the center and rising at the sides.

In regard to English roads, it is noteworthy that, having in a great degree been neglected during the war period, many are now in such a bad condition that the authorities are reconstructing them. Several of the main London roads are at present being dealt with in this

* London, England.

manner, and the writer knows of several county roads undergoing similar treatment, a stretch of one-half, longitudinally, being finished at a time, so as not to close the road to traffic. Mr. Woolley.

It is reported that the British Government, through the Road Board and the County Councils, is to assist the authorities in carrying out road works, by making grants amounting in the aggregate to about £10 000 000.

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Paper No. 1437

THE EAST CANYON CREEK DAM*

By A. F. Parker, M. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, Samuel Fortier, A. J. Wiley, Herbert E. Bellamy, and A. F. Parker.

Synopsis.

This paper describes the design and construction of a concrete dam which depends on its arched form for stability.

The East Canyon Creek Dam, on East Canyon Creek, Morgan County, Utah, is 12 miles above the Town of Morgan and 37 miles from Ogden, Utah. The reservoir formed by the dam stores water for late irrigation of lands in the Salt Lake Valley south of Ogden, and forms a part of the irrigation system owned by the Davis and Weber Counties Canal Company, of Ogden. The mean altitude of the site is 6 000 ft.

The structure is of interest because of its slight appearance and the successful manner in which it stands the test of use. The arch has a length of 128 ft., the radius of its up-stream face varying from 78 to 98.8 ft. The arched portion is 190 ft. high above bed-rock and 145 ft. above the outlet; its thickness is 26 ft. at the bottom and 5 ft. at the top. The arch abuts against a tangent of gravity section on one end, and against a tangent structure of hollow-deck type on the other. The outlet gates are of the full balanced type.

The dam was built during the winter, and the materials available for sand were of very doubtful appearance, containing much fine silt and

* Presented at the meeting of May 7th, 1919

clay. The success of the structure adds to the number of arch dams built which demonstrate and sustain the use of the simple formula for the strength of cylinders in computing the dimensions of such dams. The costs of the work are given.

General.

In 1896 a dam was built on East Canyon Creek, 58 ft. in height above the outlet; in 1900 the height was increased by an addition of 30 ft., and in 1902 it was raised to its ultimate elevation of 100 ft. The first dam formed a reservoir having a capacity of 3 850 acre-ft., and this was increased by the additions to 9 000 and 14 000 acre-ft., respectively.

The old dam was of the rock-fill type, with a steel core. With conditions as they existed at the time this dam was built it is probable that this was the best type to be adopted. It served its purpose for a long time, but finally developed a leakage which ultimately amounted to about 40 sec-ft. There seemed to be no danger of a break in the dam, but the leak became so great that it threatened its usefulness for storing water. The impossibility of repairing the leak and the need for a greater storage capacity caused the building of a new structure at the lower or down-stream toe of the old dam.

Preliminary surveys were made in 1912. The design and plans of the dam were under discussion for two years, and the contract for construction was let on April 21st, 1915. The dam was completed in June, 1916. Building roads, making plant foundation excavations, and sundry preparations consumed much time, and the first concrete was placed on September 30th, 1915. This resulted in practically all the concrete work being done in the coldest winter weather and under very trying conditions generally.

Water Supply.

The water-shed has an area of 156 sq. miles and is high, mountainous country, with an elevation of from 6 000 to 9 000 ft. The portion of the stream flow available for storage is that of the winter and the spring floods. The quantity varies greatly, according to the precipitation of the seasons. As nearly as could be determined, there was probably as much as 65 000 acre-ft. available in flush years, with a probable minimum of 20 000 acre-ft. when precipitation was light. The econo-

mical capacity of the reservoir was fixed as 28 000 acre-ft., which called for storage to the 140-ft. contour above the outlet to the old dam.

Features of the Site.

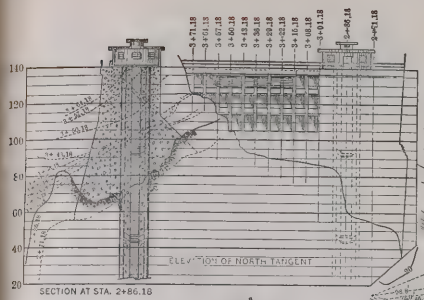
At the site of the dam a strong, prominent upheaval of conglomerate rock shows above the mountain slopes for several miles. Where the creek cuts through the conglomerate, a short, deep gorge or canyon, with solid rock walls, makes a favorable place for building a dam. The rock canyon is only about 1 500 ft. long, with the up-stream end sharply defined by vertical cliffs, and the lower end gradually tapering off to the general mountain slopes.

The rock is a limestone conglomerate, formed of fragments of rhyolite, sandstone, and limestone cemented together with calcite. The rock lies in beds of greatly varying thickness; the bedding planes are very strongly marked, and have a heavy down-stream dip. There are many soft and disintegrating spots; and some of the bedding planes have thick fillings of rotten rock and clay. Between some of the bedding planes slight stratification shows in the thinner layers, but where the planes are widely separated (sometimes as much as 40 ft.) there are massive layers of conglomerate with no stratification, and they are exceedingly firm and hard.

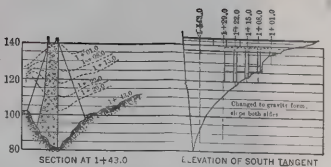
The sides of the canyon are very irregular, but, in general, are roughly vertical—in places overhanging—for the first 40 to 60 ft., and, above the vertical cliffs, the slopes fall back at varying angles. A casual inspection gives one the impression that the rock is unreliable, both as to strength and water-tightness. A close inspection shows the possibility of finding good sound rock on which to found a dam.

The conformation and quality of the rock fixed the location of the dam at the down-stream toe of the old dam. The body of the old dam precluded the adoption of the best location. It became necessary to locate the dam lower down stream, and even then the excavation cut the toe of the old dam somewhat. It is evident that extending the excavation into the toe of the old dam was a matter of difficulty and grave risk. The fact that the reservoir above the old dam was full of water and that a leak of 40 sec-ft. existed added to the difficulty and danger. That the work was done successfully is due to the absence of alluvial material. The excavated material was nowhere finer than coarse sand, but most of it consisted of coarse fragments, with many large rocks.

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1437.
PARKER ON
EAST CANYON CREEK DAM.



SECTION AT STA. 2+86.18



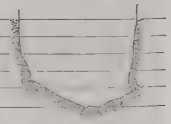
SECTION AT 1+43.0 — ELEVATION OF SOUTH TANGENT

SECTION THROUGH SPILLWAY

ARCH DATA AND QUANTITIES

CONTOUR	CHORD	ARC	ANGLE	RADIUS
140	122.2	131.5	76° 16'	98.8
130	"	"	"	"
120	"	"	"	"
110	"	"	"	"
100	"	"	"	"
90	"	"	"	"
80	122.2	131.5	76° 16'	98.8
70	101.5	106.6	61 30'	93.8
60	82.3	84.87	49 11'	98.8
50	74.5	76.72	47 46'	92.0
40	71.5	73.69	48 50'	93.18
30	68.8	70.03	49 22'	82.00
20	65.0	68.12	49 54'	78.23
10	63.5	67.76	51 44'	75.67
0	63.0	67.41	53 21'	72.34
−10	61.0	63.10	51° 40'	70.1
−20				
−30				
−40				
−50				

MAXIMUM SECTION

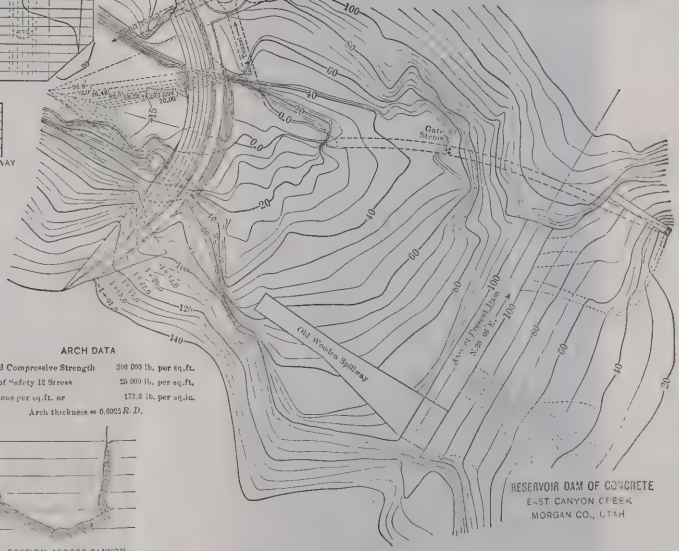


SECTION ACROSS CANYON

ARCH DATA

Assumed Compressive Strength	300 000 lb. per sq.ft.
Factor of Safety 12 Stress	25 000 lb. per sq.ft.
or 12.5 tons per sq.ft. or	173.6 lb. per sq.in.

Arch thickness = 0.0025 $R.D.$



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Bed-rock was found 45 ft. below the creek bed, where the average distance between the canyon walls was 45 ft. The surface of the bed-rock was very irregular, being cut with channels and pot-holes, but all smooth and highly polished, evidently by flowing water carrying sand and drift. Some of the pot-holes were as deep as 5 ft., and much larger at the bottom than at the top; and remains of the boulders that ground them out were found in each. No cracks or fissures were found, and the only preparation necessary to make a foundation was some roughening of the polished surface.

The Design.

The peculiarities of the site plainly indicated the adoption of a dam of the arch type. The location, above several towns and a railroad, where a break would result in a disaster of magnitude, made safety of prime importance. Calculations showed that an arch dam could be built for about one-half the cost of any other type. It was realized that the slight appearance of an arch dam of short radius, as compared with the greater bulk of gravity sections, might cause apprehension as to safety. It is probably this slight appearance that makes so many engineers reluctant to adopt the pure arch type.

From the numerous articles describing arch dams, the many discussions on their design, and the frequency with which lists of arches actually built appear in such articles, one gains the impression that a general doubt exists as to the safety of structures of such slight appearance. However, the fact that so many arch dams—many of strikingly light section and frequently subjected to trying conditions—have been built without a single instance of failure occurring, which cannot be said of any other type, is strong proof of good qualities and safety. The test of actual use shows that arches under water pressure are the safest form that can be built.

All that could be found in print relating to arch dams was studied, and the proposed mathematical treatment was applied to the design of the structure. It was found that in all cases the dimensions necessary to resist the complicated shears, bending, and temperature stresses did not exceed the dimensions computed from the simple formula for the strength of cylinders under pressure, when the unit stress on the masonry was conservative. To decide on a conservative unit stress, one must be guided by the quality of the materials, the importance of the

structure, etc. Examples are found of arch dams in successful use having stresses ranging from a minimum of 10 tons to a maximum of 61 tons per sq. ft. On account of the importance of the structure, in this case, a stress of 12.5 tons per sq. ft. was adopted.

The thickness of the arch was found by using the simple formula,

$$T = \frac{RP}{S},$$

where T = the thickness of the arch, in feet;
R = the radius of the up-stream side, in feet;
P = the water pressure, in tons per square foot;
S = the stress on the masonry, in tons per square foot.

For practical use, let the depth of water at any point be represented by D; it is easily found that $0.0025RD$ gives the thickness for a stress of 12.5 tons per sq. ft. The thickness of the arch, in this case, was finally determined by the use of this extremely simple formula.

In the hydrostatic arch, the loading is definite and symmetrically applied, as well as easily determined. The stress is primarily all compression, increasing directly as the depth of the water. Ice thrust, when present, adds a very uncertain amount of compression, and this is best provided against by the use of vertical reinforcement to distribute the stress over a larger area; but the elasticity of the masonry and the supporting rock, the changes due to variations of temperature, the combination of cantilever and gravity action, etc., necessarily produce complicated shear and bending stresses; of such stresses but little is really known, for their determination involves assumptions on which authorities do not agree, and which, in the final analysis, are only the judgments of the designers.

It is to be regretted that there is no way of determining the actual factor of safety in arched dams. All that is now possible is to apply the formulas of theory and build the structure; if it does not fail or develop signs of weakness, the calculations used in the design may be pronounced safe and proper to use in similar cases. Until some arch dam actually fails under pressure, there is nothing to show the true limit between bold and conservative design. In examining published plans of existing gravity dams, does it not sometimes become apparent that, in some instances, the higher central and so most costly part could have been made in the form of an arch with tangent ends of gravity or other

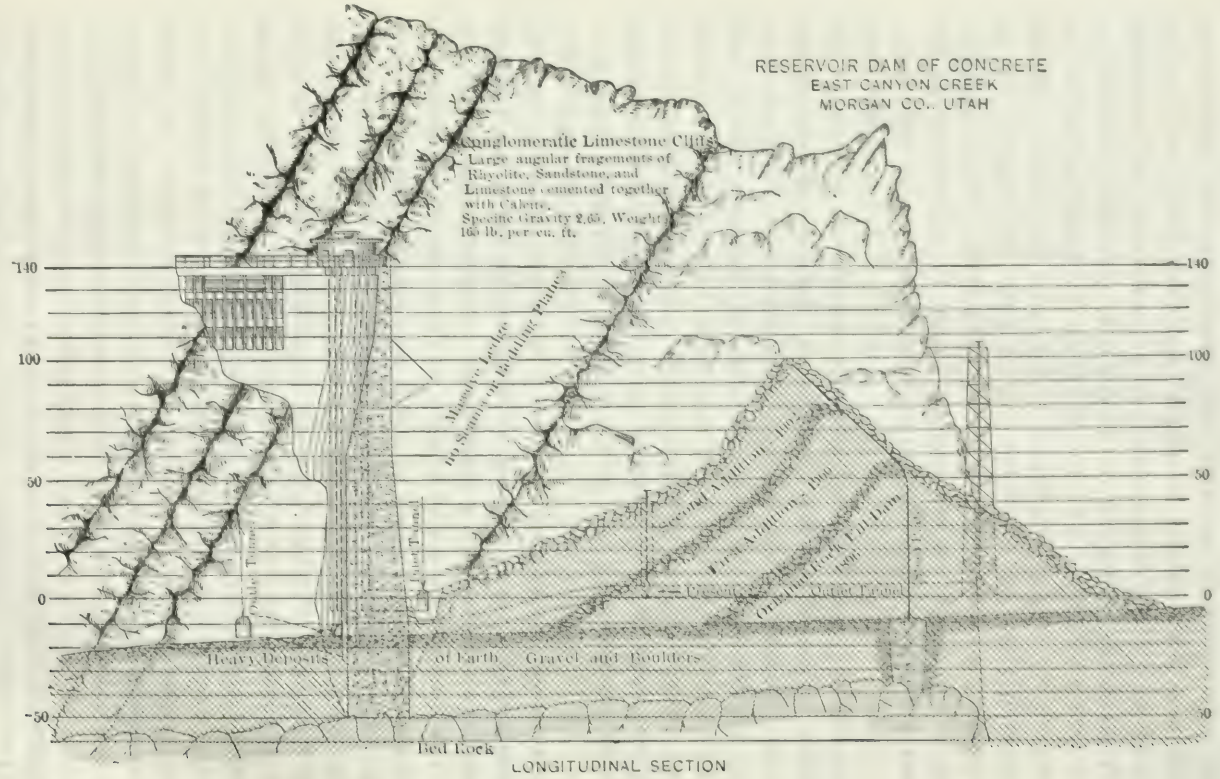
RESERVOIR DAM OF CONCRETE
EAST CANYON CREEK
MORGAN CO., UTAH
Conglomeratic Limestone Cliffs
Large angular fragements of Rhyolite, Sandstone, and Limestone cemented together with Calcite.
Specific Gravity 2.65. Weight 165 lb. per cu. ft.
Massive Ledge no Signs of Bedding Planes
140
100
50
0
-50
Heavy Deposits
of Earth Gravel and Boulders
Bed Rock
LONGITUDINAL SECTION

FIG. 1.

type, according to conditions, without detriment to safety and at considerable saving of cost? The saving in the quantity of foundation excavation alone would be considerable.

It is always desirable to use as little concrete as is consistent with safety. It is variously stated that the minimum quantity of concrete is needed for a given thickness of dam when the central angle of the arch includes about 133°, when the versed sine is one-third of the chord, or when the radius is equal to the chord multiplied by 0.54. In this case it was found impractical to consider the element of the minimum quantity of concrete; it involved producing overhangs in the faces of the arch, and it is probable that cases are rare where it is practical to maintain a constant angle between the limiting radii. Recorded examples of arch dams show that in some cases local increases of thickness have been made in order to give increased strength; in this case no such additions to the calculated thickness was considered necessary, but in places the joining of the arch to the cliffs had small increases of thickness in order to cover cracks and soft places in the rock, and for no other reason.

When the arch is very thick, compared with its length, the matter of arch action is very uncertain. It is known that a structure loses its arch characteristics when the thickness at the crown is too great, compared with the radius and length of arch, but at what point arch action ceases is very uncertain; one authority states that a curved dam will act as an arch if the thickness does not exceed one-half the radius, and another says one-third. It is probable that in this case the bottom part of the arch acts more as a bulkhead or plug to the canyon, than as an arch.

The sides of the canyon forming the abutments of the arch have such direction as to be nearly parallel with the center line of the included arch angle, instead of being on radial lines, as is to be desired. To cut into the rock sides to give abutments on radial lines was expensive, and, on the north side, such blasting of rock was too near the tunnels and gate-shaft for safety; the shaft was located close in, for reasons of economy. Therefore, it was decided to trust to the key afforded by the inequalities of the rock—in this instance unusually favorable for a good key—without blasting out any more rock than was necessary for finding good solid abutments, notwithstanding the fact that the line of the rock made an angle with the line of the

resultant of the arch thrust equal to the full angle of repose for masonry. There was placed over such abutments steel dowel pins, of 1-in. square twisted pieces, 4 ft. long, placed one-half in the rock, in grouted holes, and the remainder built into the concrete, one dowel to every 3 ft. vertically and horizontally. Such dowels may not do much good, but they certainly give a look of security.

To fit the arch to the cliffs properly, it was found necessary not only to adopt radii varying in length from 70 to 98.8 ft., but, at Elevation 60, to change the batter from the up-stream to the down-stream side. The batter begins on the up-stream side of the dam at 10 ft. below the level of the old outlet, assumed for convenience as Elevation 0.0, so that the point 10 ft. below has a minus value. The down-stream side is vertical to Elevation 60, where the batter changes to that side, and is continued thence to the top. The calculations for thickness bring the dam section to a point at the top; but the top thickness is assumed in this instance at 5 ft., and the surface batters downward to meet the other batter at Elevation 110, where the thickness is 7.41 ft. More for appearance than anything else, there is a rib at the top, on the down-stream side, of 8-in. projection and 4 ft. deep. On the up-stream side there is a parapet wall, 3.5 ft. high, and opposite is a railing of 1½-in. pipe supported by posts of 3-in. I-beams; provision is made for the expansion and contraction of the pipe.

The length of the dam over all is 273 ft., the south tangent being 43 ft., the arch 128 ft., and the north tangent 102 ft. Up to about Elevation 60 the arch is not more than 65 ft. long, mostly only from 45 to 50 ft. The excavation for bed-rock extended to 45 ft. below the creek bed, or 50 ft. below the old outlet, which was assumed as the zero of elevations. The total height of the dam is 190 ft. From bed-rock to Elevation — 10 the thickness is constant, because of balanced pressures; the use of forms began at Elevation — 10, with a dam thickness of 26.25 ft., but to that level the thickness is from 28 to 30 ft., for in the foundation timbering only the heavy timbers were removed, and the whole space between the lagging was filled with concrete.

The length and radius of the arch was determined by the set of the tangent ends. It was necessary to use tangent ends, for the calculations showed that otherwise a longer radius was needed, and then the ends of the arch would be too nearly parallel with the sides of the canyon; in fact, the dimensions and form adopted showed that the

cost of building would be less than with any other form tried. The most difficult matter involved in the design was to find a balance of the elements involved, at the same time having due regard for economy and sound rock for abutments. This was only accomplished by patience and many trials of different forms and locations.

The Tangent Ends.

The south tangent of gravity section was first designed as a deck section, but after the foundation to receive it was blasted out, the contour was changed so much that the gravity section with slopes on both sides was substituted. On the north side the rock fell away in such a manner that a proper gravity section could not be built on it where the slope of the structure is down stream, and it became necessary to adopt a hollow-deck section where the sloping side could be up stream. The deck is formed of arches below and a straight surface above, set on piers 7 ft. from center to center. The foundations of the tangents, perched on the sides of the cliffs, had dowel pins of the same kind as those used in the arch abutments; there were plenty of them, and they were set into the rock before the concrete was placed.

As it was necessary for the tangent ends to receive the thrust of the arch and carry it to the rock sides of the canyon, the hollow spaces in the north tangent required special treatment. Accordingly, two heavy braces or ribs were placed, reaching from pier to pier and to the side of the canyon. The lower brace, at Elevation 115, is 7 ft. wide and 4 ft. high, with a reinforcement of 36 rods of 1-in. square twisted steel; the upper one, at Elevation 122, is 6.25 ft. wide and 3 ft. high, with 20 rods of the same size and kind; the horizontal rods in both beams are surrounded with hoops of ½-in. twisted steel every 12 in. The working floor of the spillway provides for the thrust at the top.

The Spillway.

The spillway, 42 ft. wide and 5 ft. deep, is in the north tangent. It is fitted with top-hinged, swinging, steel supports or needles, 3.5 ft. from center to center, for supporting ordinary plank flash-boards. It is intended that the ordinary operation of the flash-boards shall be by hand, using hooks on the irons placed for that purpose on the boards. For emergency use means are provided for prying up the needles by using a steel bar kept for that purpose, until the bottom passes the

lower support and allows the needles to swing down stream, thus releasing the boards supported by the needles. In such event the boards released go down stream, and may be lost; but the need for such release is infrequent, and the loss of a few boards of little moment. The spillway apron is from the top of the upper thrust beam to the crest of the spillway; it is arranged to throw the water well away from the dam, where it finds its way to the creek channel without further provision for directing it.

Contraction Joints.

There are only two contraction joints, and they are placed where the arch abuts the tangents. The north joint begins at Elevation 60 and the other at Elevation 80. One side of the joints was built in advance of the other, and a coating of hot, DX-grade, California paving asphalt was applied to the surface before the remaining concrete was placed. Across the joint, and 2 ft. back from the up-stream side, a 1½-in. pipe was placed, one half in each side; when these pipes were removed, a 2-in. hole was left, and this was run full of the hot asphalt, as additional security against leakage. The joints, being on true radial lines, had no dowels or reinforcing steel extending across; but heavy offsets were made in the concrete.

Steel Reinforcement.

No reinforcement was used until Elevation 50 was reached; from that point to the top, 1-in. square twisted steel was placed in 2-ft. squares and 1 ft. back into the concrete on both sides of the arch. The gravity section tangent has no reinforcement, but the deck type of tangent has a reinforcement of ¾ and ½-in. square twisted steel rods. It was decided to use the steel in the manner described as a compromise of apparent demands of the elements involved—the horizontal steel to stiffen the thinner parts of the arch, the vertical to distribute the ice pressures, and all to prevent cracks. It certainly prevented cracks, but more steel may have been used than was strictly necessary. The heavy reinforcement may be warranted in the interests of safety, especially considering ice thrusts, for ice of great thickness forms here.

The gate-house, parapet wall, and gate-tower had a reinforcement of ¾ and ½-in. square twisted steel in all places where its need was evident. The item of structural steel is covered by railings, trash-rack, gate-house doors and shutters, and the spillway fittings. About 90½ tons

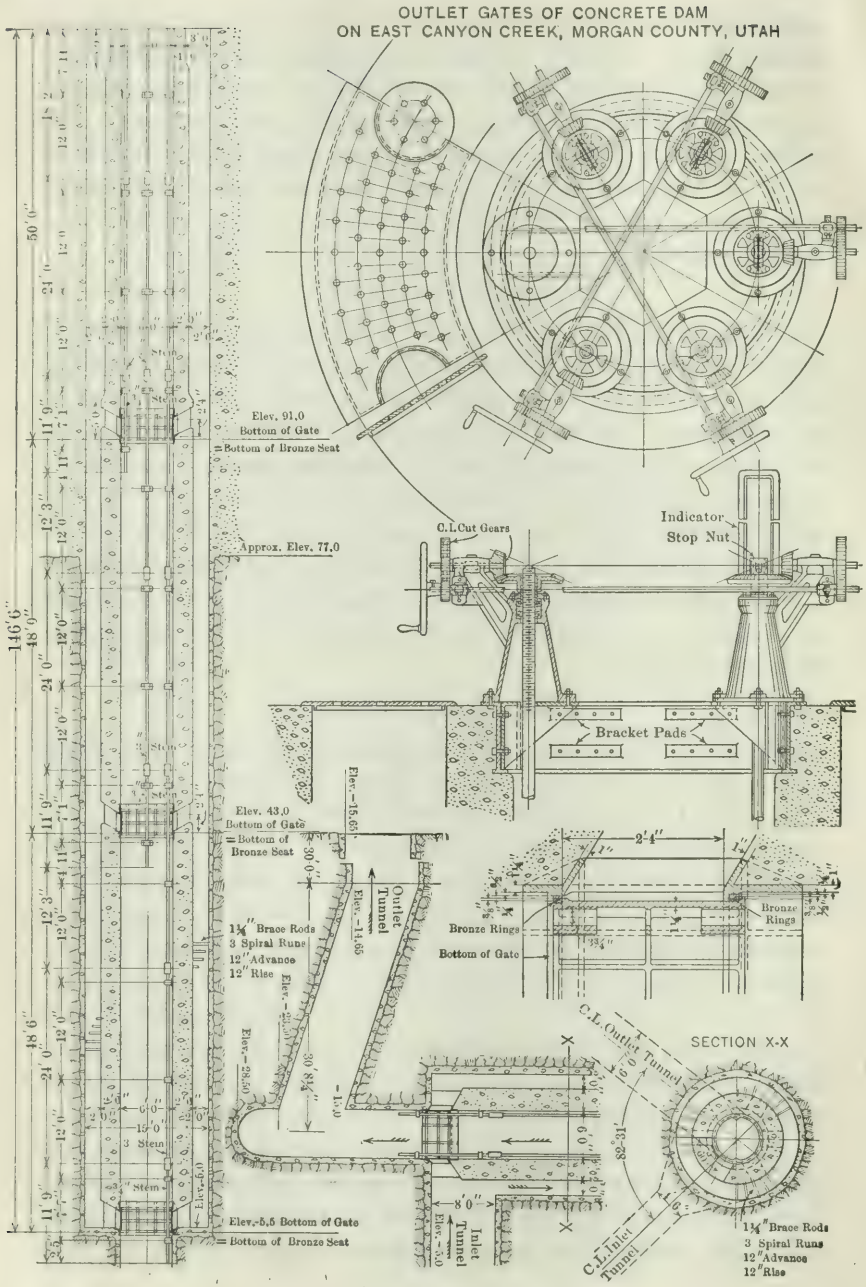
OUTLET GATES OF CONCRETE DAM
ON EAST CANYON CREEK, MORGAN COUNTY, UTAH
Elev. 91.0
Bottom of Gate
= Bottom of Bronze Seat
Approx. Elev. 77.0
C.L. Cut Gears
Indicator
Stop Nut
Bracket Pads
Elev. 43.0
Bottom of Gate
= Bottom of
Bronze Seat
Elev. -15.65
Outlet
Tunnel
Elev. -14.55
1¼" Brace Rods
3 Spiral Runs
12" Advance
12" Rise
Bronze Rings
Bottom of Gate
Bronze
Rings
SECTION X-X
C.L. Outlet Tunnel
C.L. Inlet
Tunnel
Inlet
Tunnel
Elev. -5.5 Bottom of Gate
= Bottom of Bronze Seat
Stem
146'6"
50'0"
48'0"
18'6"
11'9"

FIG. 2.

of reinforcing steel were used. The greater part of this was in the arch, but a very considerable quantity was used in dowel pins, braces to the gate-tower crossing the surrounding space, as well as in the tower proper and about the gate-house, spillway, and thrust beams; no accurate segregation was made, as all was paid for at one price per pound, regardless of kind or place of use.

The Outlet System.

The outlet system comprises an intake tunnel, a discharge tunnel, a vertical gate-tower in a shaft and carrying three balanced gates, set at the bottom of the tower and at the one-third and two-thirds points of the height, a sump or water cushion at the bottom of the gate-tower, and a hand-operated device in the gate-house for opening and closing the gates.

The concrete tower carrying the gates has an inside diameter of 6 ft.; the walls are 30 in. thick, and the outside diameter is 11 ft. This gate-tower is set in a shaft, 15 ft. in diameter, blasted out of solid rock up to about Elevation 70 and thence to the top forming a part of the north tangent. The annular space about the tower is always full of water, supplied through the intake tunnel; and, on opening a gate, discharge occurs into the 6-ft. opening in the tower, from which it drops to the sump and is discharged thence to the stream through the outlet tunnel. Leading from the reservoir to the annular space in the shaft about the gate-tower is the intake tunnel, 60 ft. long, with a cross-sectional area of 33.8 sq. ft., and with its bottom at the level of the bottom of the lower gate, at Elevation — 5, or 145 ft. below the top of the dam.

The outlet tunnel is also about 60 ft. long, and has a cross-sectional area of 50.2 sq. ft. It is set at a lower elevation than the inlet, and is arranged for a water cushion in the central shaft and a water seal for preventing a draft of air through the gate-tower, thus precluding the formation of ice. This is a matter of importance in this cold place. As it is necessary to draw all water through the old outlet system when the reservoir surface is below Elevation 100, the tunnels were designed to carry only the capacity of the old outlet. It would have been better, perhaps, to have made them larger, in order to avoid excessive velocities with the larger flows possible when the water is higher than the old dam.

The Trash-Rack.

The old outlet had been provided with a light steel trash-rack, and, as it had given no trouble during 18 years of use, a similar rack of ½ by 5-in. steel bars, 5 in. from center to center, was placed at the new outlet. The rack rests on four pieces of 90-lb. rails, spanning the shorter way. When the water was drawn off, in the fall of 1917, it was found that trash which had gathered on the rack had reduced the flow of water until the resulting pressure had bent the steel rail supports and bent and twisted the rack bars. It is fortunate that the rack was not stronger; had it been equal to holding the pressure, a very embarrassing situation would have resulted. No indication of the condition was visible, and it was not suspected until the water was drawn off.

The bent and twisted rack was entirely removed, and the channel between the old and new outlets, about 5 by 5 ft., with an open top, was covered with a grill of pine poles having a minimum diameter of 9 in. These poles have an average length of 16 ft., and span the channel in an inclined position, extending from the outer side of the channel to the rock wall of the cliff on the other side, with clear spaces of 6 in. between the poles. The distance covered is 70 ft. The poles are fastened in place by setting 1-in. steel pins in the rock and using ties of No. 9 galvanized wire. This arrangement has not yet been tested, but promises good results.

The Cylindrical Balanced Gates.

Each gate works in a cast-iron ring or cylinder set in the concrete walls of the gate-tower. Each cylinder has six 2 by 2-ft. openings in its perimeter, and these are symmetrically spaced. Freely movable vertically inside of the stationary rings are the gates proper, which are cast-iron cylinders with no openings. When the gates are fully down, the openings in the outer cylinders are closed. At the tops and bottoms of the cylinders are fitted heavy rings of naval bronze; when the inclined faces of the bronze come in contact, a water-tight joint is formed.

This arrangement insures a perfect balance of water pressures. Use has demonstrated that no more force is required to operate these gates under pressure from full heads of water than when there is no pressure. Hand-wheels are all that are needed for operation; the arrangement

provided for the purpose works very easily, and with such smoothness and speed as to call for nothing better; power operation would be of no advantage.

There are two stems, of $3\frac{3}{16}$-in., cold-rolled steel shafting, rigidly attached to opposite sides of each gate, extending up to the operating gears and below each gate, to form guides to steady them and assist in keeping them in position. The stems of the lower gates pass through holes in the castings of the upper gates, and all are provided with bronze-lined guides every 12.5 ft. in height. The top guides have feather keys, in order to prevent turning, and the bronze linings have a clearance of $\frac{1}{4}$ in. to allow for incrustations. The upper gates, when opened, discharge through the lower gates, and the small projections of stems and guides do not interfere with the discharge.

The tops of the stems are threaded with single square threads, three to the inch, working in heavy bronze nuts with thrust ball-bearings above and below the nuts. The hoists are arranged to operate the two stems of each gate at the same time by using one hand-wheel, having a gearing of cast iron with machine-cut teeth. The stands carrying the gearing are set on a heavy cast-iron ring with brackets, in such a manner as to leave a large opening at the top of the shaft for supplying air to overcome the strong suction caused by the discharging water. The need of this is shown by the fact that, when the gates are discharging, it is very difficult to open the steel door to the gate-house against the air pressure, notwithstanding the generous supply of air through the top ventilator on the gate-house provided for that purpose.

The wisdom of making the stems much heavier than called for by their computed tensile strength is shown by the accidental lodging of a 5-ft. length of 6 by 8-in. timber in the lower gate in some unaccountable manner. An attempt to close the gate showed that something was wrong, for before one man had nearly exerted his full strength on the hand-wheel, the heavy cast-iron top ring was started from its anchorage. When the reservoir was emptied, it was found that the timber was cut at least half way through. Lighter stems could not have withstood such abuse without being damaged.

The Concrete.

The Red Devil brand of cement, manufactured at Devils Slide, Utah, was used throughout. The cement passed the tests satisfactorily,

and it was unnecessary to reject any. A complete apparatus for testing cement and concrete materials was provided at the work, one man being constantly employed on tests.

The sand available had a very forbidding appearance, for it had a large content of fine silt and clay, and, when made into a cement mortar, presented a "muddy" consistency, but tests showed a fair strength and a good degree of water-tightness. Crushed rock was used for the coarse aggregate.

All the tests for strength were made by breaking mortar briquettes in a tensile testing machine. It is fully realized that such tests are properly only tests for cement combined with sand as mortar, and not true tests of concrete made with such mortar and under compression. The true test for concrete should be made in a compression machine of sufficient capacity to crush 8 or 10-in. cubes or cylinders made of the concrete used on the work, including the coarser aggregate. It is probable that the cost, large size, and weight of such compression machines is a bar to general use, as was the case in this instance.

Tests of mortar composed of the cement and sand showed a satisfactory increase in strength with age, some breaks in specimens of 1 cement to 3 sand running as high as 500 or 600 lb. per sq. in. at the age of 3 months.

Briquettes were made from the mortar from the mixer nearly every day concrete was placed. No breaks could be made at 2 days. At 7 days the tensile strength showed an average of about 100 lb.; at 28 days it was always more than double the strength at 7 days; and at 18 months the minimum was 310 lb. and the maximum 645 lb., with the average running about 500 lb. Breaks were made frequently between the 28-day and 18-month periods, and in no case was there a failure to show an increase of strength with time.

Notwithstanding the bad appearance of the sand, containing as it did much earth, clay, and silt, the outcome is very satisfactory, although more cement was used than would have been the case had clean sand been available. It is now known, however, that this concrete is more water-tight than if made with all screenings. All the concrete was mixed with more water than desired, in order to help in placing it in the forms and avoid the formation of rock pockets without mortar. It is well known that with fresh concrete, made with clean sand, the cement will be washed away by the least contact with water. Some concrete

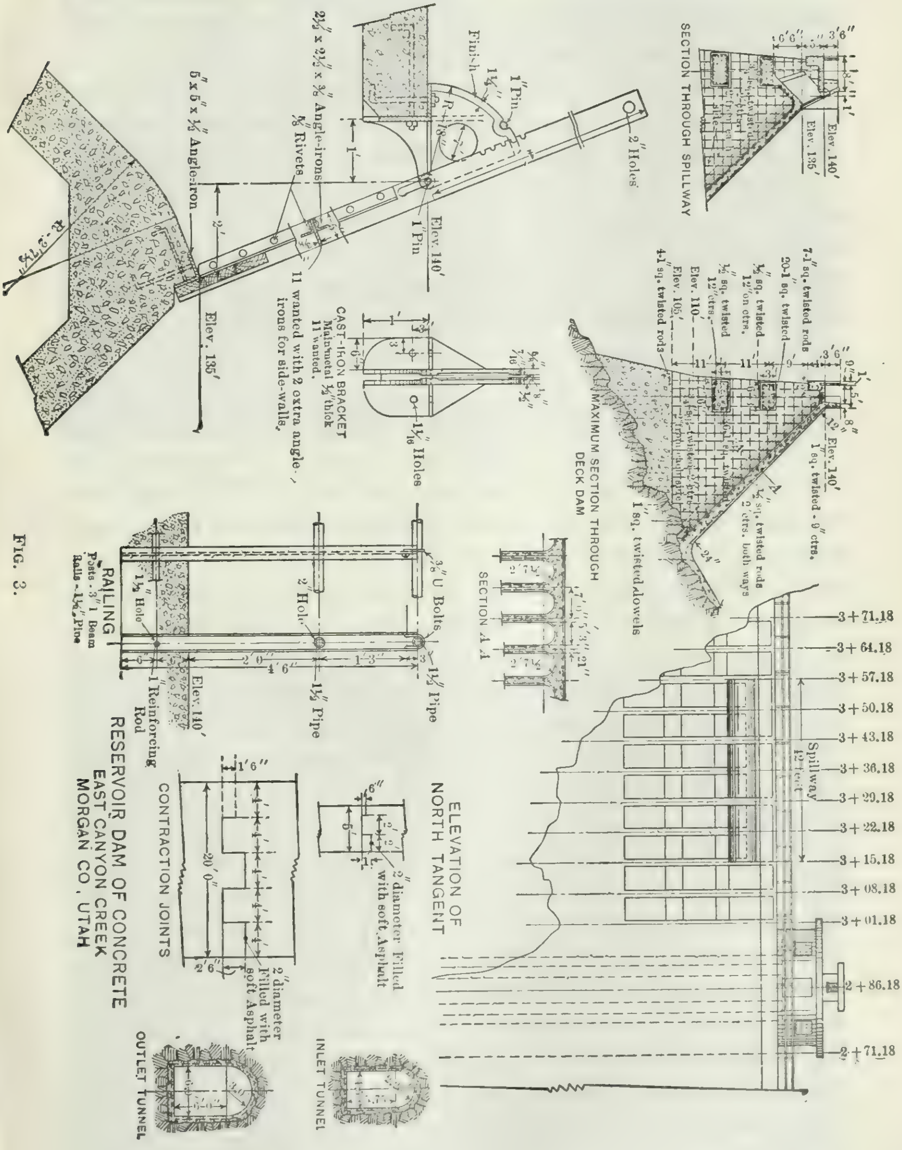
SECTION THROUGH SPILLWAY
Elev. 140′
Elev. 135′
MAXIMUM SECTION THROUGH DECK DAM
7-1″ sq. twisted rods
20-1 sq. twisted
½″ sq. twisted 12″ on ctrs.
½″ sq. twisted 12″ ctrs.
Elev. 110′
Elev. 105′
4-1″ sq. twisted rods
1 sq. twisted - 9″ ctrs.
¾″ sq. twisted rods 2′ ctrs. both ways
1″ sq. twisted dowels
SECTION A-A
ELEVATION OF NORTH TANGENT
Spillway 124 feet
3 + 71.18
3 + 64.18
3 + 57.18
3 + 50.18
3 + 43.18
3 + 36.18
3 + 29.18
3 + 22.18
3 + 15.18
3 + 08.18
3 + 01.18
2 + 86.18
2 + 71.18
2″ Holes
1″ Pin
Finish
Elev. 140′
2½″ x 2½″ x ⅜″ Angle-irons
⅝″ Rivets
5″ x 5″ x ½″ Angle-iron
Elev. 135′
11 wanted with 2 extra angle-irons for side-walls.
CAST-IRON BRACKET
Main metal ½″ thick
11 wanted.
1⅛″ Holes
⅜″ U Bolts
2″ Hole
1½″ Pipe
1½″ Hole
1 Reinforcing Rod
Elev. 140′
RAILING
Posts - 3″ I Beam
Rails - 1½″ Pipe
2″ diameter Filled with soft Asphalt
CONTRACTION JOINTS
2″ diameter Filled with soft Asphalt
INLET TUNNEL
OUTLET TUNNEL
RESERVOIR DAM OF CONCRETE
EAST CANYON CREEK
MORGAN CO. UTAH

FIG. 3.

in the foundation was placed in water without harm, or any loss of cement, and several times, when the pumps stopped, the surface of fresh concrete was flooded without bad effects, which would not have been the case with concrete made with clean sand.

The proportions of the concrete used in the body of the dam were, as nearly as possible: 1 part cement; 3 parts sand, with some screenings through a ¾-in. screen; and 5 parts broken rock, 3-in. maximum size.

In the body of the dam 4.75 sacks of cement per cu. yd. of concrete were used, and for about 800 cu. yd. about the gate-tower, for tunnel lining, and in the gate-house, the quantity was 6.25 sacks per cu. yd. It was planned to use plum rock in the concrete of the dam body, but cold weather prevented, as it was too difficult to remove the snow, ice, and dirt from the rock.

Concrete in Cold Weather.

Most of the sand was hauled early, and was in a large pile on the side-hill above the bins; and becoming wet from the fall storms, froze to a depth of 2 or 3 ft. The broken rock was coated with frozen earth, snow, and ice. Notwithstanding the fact that the bins were roofed over, the materials often froze together. Nothing was done to overcome these conditions except to mix the concrete with hot water. The concrete as it came from the mixer was just above the freezing point, and, to maintain it at this temperature, it was necessary to keep the water hot and throw out the larger, solidly frozen lumps.

Experience soon showed that the rise of temperature due to chemical changes in setting prevented damage from frost when the concrete in the forms was covered at night. Canvas was spread over the forms, steam jets prevented surface freezing, and the temperature of the mass below the surface soon increased sufficiently to remove any frost occurring there, if not too great in quantity. The steam caused heavy deposits of frozen moisture on the forms; when this melted it caused a very sloppy condition, and so the steam jets were abandoned and salamanders burning coke were substituted, with an improvement of conditions. When concrete was placed on colder days the surface of the fresh concrete often immediately froze over to the depth of an inch or two. It was found that placing fresh concrete thickly over such a surface, quickly thawed it out without harm; but when the freezing was too severe, the frozen part was picked, broken up, and thrown out. Steam

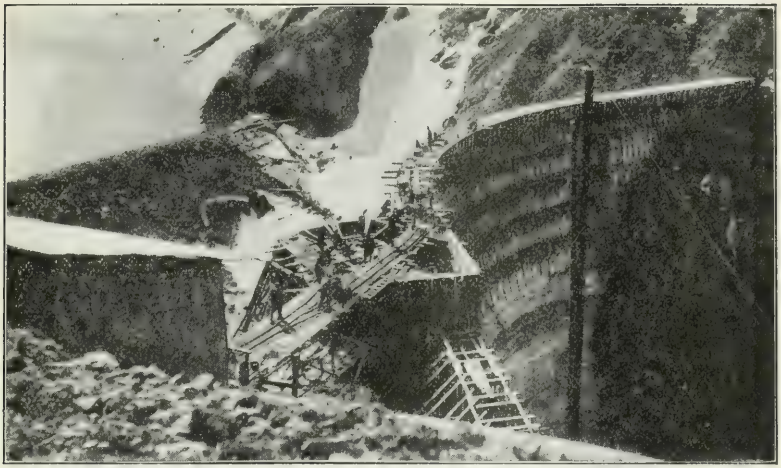

FIG. 4.—EAST CANYON CREEK DAM: NEW ARCH DAM UNDER CONSTRUCTION, OLD DAM ON LEFT.

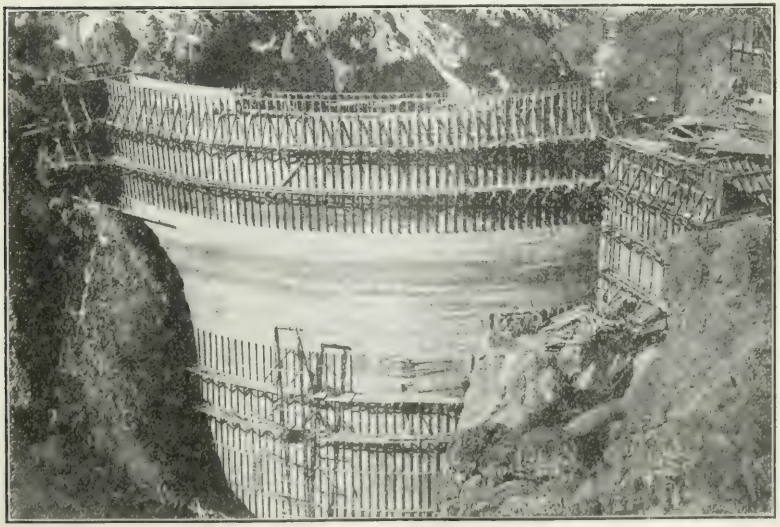

FIG. 5.—FORMWORK OF ARCHED SECTION, PARTLY STRIPPED, EAST CANYON CREEK DAM.

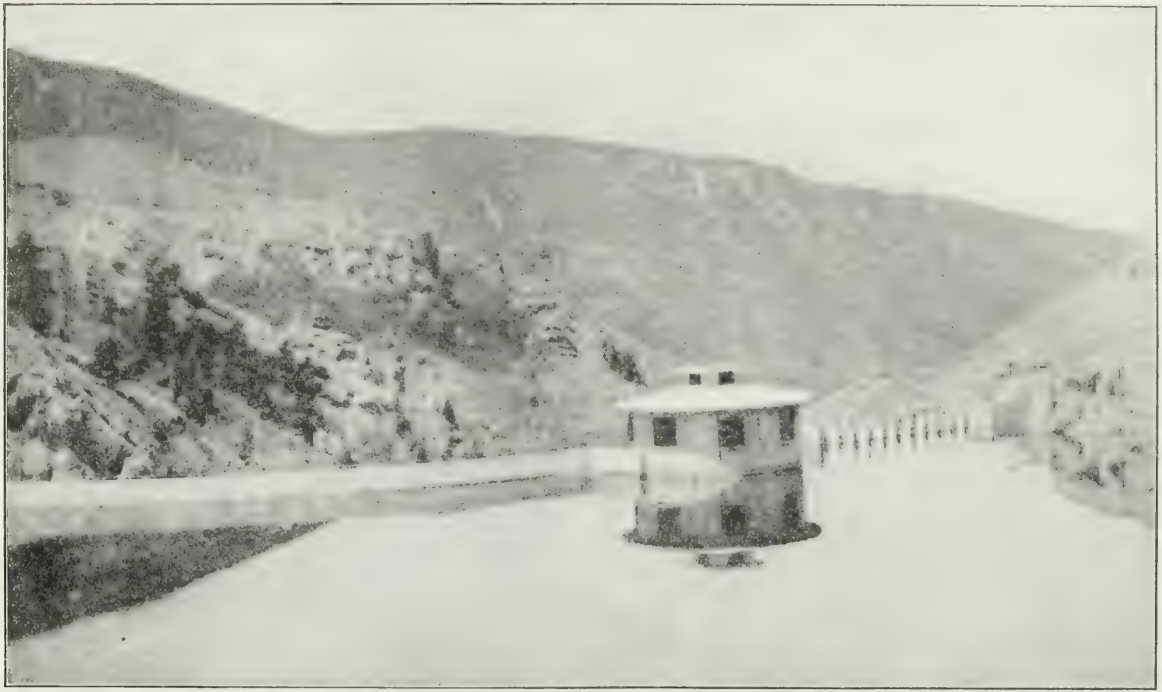

FIG. 6.—GATE-HOUSE AND WASTEWAY, RESERVOIR FULL, EAST CANYON CREEK DAM.

jets were used freely to thaw out and remove the ice from the forms, concrete surfaces, and rock walls, immediately before placing concrete against them.

The removal of the 1½-in. pipes in the contraction joints afforded a very favorable means of testing the rise in temperature in the concrete when setting. Tests of temperature by lowering a thermometer into the holes showed in every case a considerable rise, a typical test showing a temperature of 70° Fahr. at a depth of 15 ft. when the air was at 14° Fahr., a difference of 56 degrees.

The Forms.

The use of forms began at Elevation — 10. Up to that point the excavation was filled to the sheeting. The forms were built in sections 10 ft. high. At the top and bottom of each form 2 by 10-in. plates were placed, and vertical pieces of the same dimensions were set between as studs, 2 ft. from center to center, with 1 by 6-in. sheeting on the studs. At the jointing of the sections horizontal 2 by 6-in pieces were spiked to each stud, projecting 4 ft., with a 1 by 6-in. brace from below. These brackets formed a place for a working scaffold, and braces upward from the end helped to place the next section of forms.

When a form was filled to the top with concrete, and before it had set, 1-in. steel pins, 4 ft. long, with an eye at the top, were driven into the concrete, about 6 ft. in from the forms. Ties of 4-ply, No. 9 wire, were strung through the eyes in the pins and fastened to each stud at their one-third points of height, thus being inclined sharply upward. The wire ties were then twisted to tightness, and served to hold the forms true against the pressure of about 2 ft. of freshly deposited concrete; the concrete was accordingly placed in layers of about that thickness, moving back and forth as necessary to avoid excessive pressure and the resulting bulging of forms. No wires were allowed to reach from side to side of the dam. Where the thickness of the dam was greatest, two rows of anchor-pins were used, and only one row when the distance between the forms became less. Near the top, in the narrower space, the use of two rows was resumed, with the stay-wires crossing in order to keep proper distance from the forms; at the very thinnest part the wires were attached to the reinforcement. The forms about the gate-tower, spillway, and gate-house presented no special features, being of the usual 2 by 6-in. studs, 1-in. sheeting, and wire ties. Figs. 4, 5, and 6 show the formwork and the complete dam.

The Plant.

The plant presented no unusual features, for it consisted only of ordinary steam-driven machinery, and was all adapted to use on this work from such as the contractors had left on hand from previous jobs. The excavation was dewatered by three 8-in., belt-driven, centrifugal pumps. Rock drilling was done by machine drills driven by compressed air. Compressed air was also used to drive some of the hoisting engines for the derricks. Rock crushing was done by medium-sized jaw crushers, and all materials were handled by gravity. Concrete was mixed in ¾-yd., Smith mixers, and conveyed by metal-lined chutes to steel mining cars running on a light track on top of the forms.

Cost of the Work.

Table 1 is a detailed statement of the cost of the work, with quantities and unit prices.

TABLE 1.—Cost of the Work.

Tunnel and Shaft Excavations:					
Neat section	860.4 cu. yd.	@	$5.40	$4 646.16	
Overbreak	123.1 cu. yd.	@	2.16	265.90	$4 912.06
Foundation Excavation, above Elevation —35:					
Neat section	1 623.9 cu. yd.	@	$2.50	$4 059.75	
Overbreak	825.8 cu. yd.	@	1.00	825.80	4 885.55
Foundation Excavation, below Elevation —35:					
Neat section	547.7 cu. yd.	@	$5.00	$2 738.50	
Overbreak	53.6 cu. yd.	@	2.00	107.26	2 845.70
Hauling Cement: 19 170 bbl. @ 87½ cents					16 773.75
Reinforcing Steel: 181 200 lb. @ 3½ cents					6 342.00
Concrete: 15 888.38 cu. yd. @ $5.25, including forms					83 414.00
Cement: Furnished by the owners					35 742.24
Concrete Lining: tunnels and gate-shaft, 590 sq. yd. @ $1.45					885.50
Balanced Cylinder Gates					6 250.00
Sundry on Contract: Force account				$8 713.70	
Lumber left in excavation				450.00	
Extra formwork				500.00	
Structural steel				938.98	10 602.68
Inspectors on Concrete					778.15
Lands Purchased for Reservoir					3 423.80
New Road Around Reservoir					5 278.60
Total					$182 134.03
Engineering, 6.15%					11 217.24
Total Cost					$193 351.27

The cost per acre-foot was $6.90. The cost of the concrete was $6.37 per cu. yd. The engineering cost covers all preliminary work from the beginning.

Conclusion.

In the season of 1916 the water in the reservoir was raised only to the 100-ft. level, for the concrete work was too new to be put under full pressure. In 1917 the dam was full to the top, and a heavy flow wasted over the spillway. It is estimated that the water-shed furnished some 60 000 acre-ft. in that season. During the season of 1918 the entire yield of the water-shed only filled the reservoir to the 120-ft. contour. The dam has thus been fully tested. The gates have passed at times as much as 600 sec.-ft. without shock or jar. In the fall of 1917 the reservoir was completely emptied and the structure carefully inspected. Not a single crack or defect was visible; no sign of weakness, incipient disintegration, or scaling of concrete could be found. The sump was baled out, and it was found that the water in heavy flows falling from a height had not injured the concrete or rock in the slightest degree, the only thing needing correction was the trash-rack, as already mentioned. When there is no water back of the dam, the contraction joints open about $\frac{3}{8}$ in., but, as the water rises, the joints gradually close tight.

Although resting on good, solid, water-tight rock, and joining it on the sides, there were several bedding places and cracks that threatened leaks around the dam. It was not attempted to stop these, because small leaks could do but little damage, beyond loss of water, and it was decided to try them out. As the reservoir was filled, and when the water was about 70 ft. in depth, two leaks appeared on the walls of the canyon, on the south side and about 75 and 125 ft., respectively, down stream from the dam. The combined flow of the two leaks is estimated at 2 sec-ft.; it is impractical to measure them. When the dam was first put under water pressure there was a heavy seepage or percolation through the concrete, heaviest at the horizontal joinings between concrete forms, but all rapidly reducing in quantity until the percolations were so small that they only showed when in the shade; when the sun shone on the surface it became dry. There was not one leak through the dam or its joining with the rock, nothing more than the percolations mentioned.

The seepage brought to the surface salts from the cement, and they cause a streaked efflorescence on the back of the dam; this is very plainly in evidence, and presents a false appearance of leaks that is very misleading, especially in a picture. That there are no leaks is shown by

the fact that when the gates were first closed, and until the appearance of the two small leaks mentioned, the flow of the creek was reduced so much that the Fish and Game Commissioner demanded that more water be admitted in order to keep the fish from being destroyed. Accordingly, a gate was opened a little.

When the dam was built it was necessary to leave a hole, 2 ft. square, through it at the lower gate level. This was stopped temporarily by a timber bulkhead. It is planned to place a 14-in. pipe in this opening, and, at the down-stream end, a heavy valve. The latter will be covered with a concrete house to guard against frost and meddlers.

As it is important to know the exact depth of water back of the dam, and its rate of rising or falling, a special device for this purpose has been designed. A heavy copper float, guided by two copper wires set vertically in the annular space about the gate-shaft, is suspended by a plated steel tape; the latter is wound on a 28-in. drum geared down by two reductions to the point where 50 ft. of movement of the tape will cause a movement of 1 ft. on the balancing weight. A wooden rule, 1 ft. long, is attached by the side of the tape to read the fractions of feet. It has not yet been tested, but promises to give good indications of small variations in depth, as this information is most important in the operation of the reservoir.

In spite of the unfavorable conditions of weather under which the dam was built, the fine clayey sand used, and the slight appearance of the arch, it may be pronounced a very successful structure. The contractors are entitled to credit for their faithful prosecution of the work under adverse conditions, and for their patience, courage, and fidelity to the interests of good work. The structure was not plastered or patched, but remained just as the forms left it, and with no deviation from true lines; no places show in the least careless or hasty work.

Credit is due to Samuel Fortier, M. Am. Soc. C. E., who, being familiar with the old dam and the conditions of the site, endorsed the building of the new structure. He also gave much valuable advice about its design.

The work was done by the Utah Construction Company, of Ogden, Utah, Mr. A. J. Sartoris, of Oakland, Cal., being directly in charge as Superintendent. Mr. H. F. Irwin was the writer's representative on the job.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—Mr. Parker has shown wisdom in selecting a low unit stress for the concrete of this arch dam. Reduced to square inches, this unit stress is 174 lb. It is true that there are venturesome souls who would use concrete stresses as high as 850 lb. per sq. in., as Mr. Parker states, and these men call themselves engineers because they spend one dollar where others would "squander" several.

From the achievements of structural steel designers, in which factors of safety are sometimes about two or less (the elastic limit being considered the point of ultimate usefulness), an argument might be constructed for higher units in concrete, which would look plausible enough on its face. Steel and concrete, however, are quite different in their behavior.

If a dam begins to show signs of weakness, it does not help matters to post notices to reduce the loads, as in the case of failing or inadequate buildings. There are many buildings and other structures which never receive the loads they are supposed to carry. Some of them would collapse if they did.

There are many arguments for conservatism in the selection of unit stresses in the design of a dam, chief of which, of course, is the dire consequence of its failure. Next comes the argument as to the certainty of the realization of the load (already hinted at). Other reasons for low unit stresses will be mentioned later.

Unit stresses in dams received little notice in the days when gravity dams were built almost exclusively. Much was written some years ago concerning shear in a gravity section, but this was in the days before under-pressure began to be recognized as a factor to be considered in the stability of dams and a cause of most failures. If a gravity section is properly designed for under-pressure, in construction joints and beneath the dam, vertical shear in wide-spreading toes need give no concern, for those toes will have no existence.

When hollow dams and multiple- or single-arch dams began to be built, more care was necessary in the matter of unit stresses. Even then, and before and since that time, the one great essential in design was often neglected, namely, the applied loads. It would seem to be a simple matter to know and provide for the pressure of water, for that pressure is so sure, and obeys laws that are quite well known. In spite of this, there is not a book in the English language (excepting very recent ones) that even mentions the great force of water in horizontal joints and at the base of a dam. Consequently, an engineer is at a disadvantage in designing a dam, especially when he sees elaborate calculations to find the true theoretic cross-section, that

* Pittsburgh, Pa.

Mr. Godfrey.

ignore entirely forces of tremendous moment which render these calculations utterly void. Mr. Parker's paper does not state that he considered under-pressure. It is true that this does not affect the stability of an arch dam so vitally as it does that of a gravity dam or a hollow dam; but, if the section of an arch dam should be so light that its weight would approach that of a prism of water conceived to be constructed on its base, a danger point is reached where the dam is likely to be raised from its base and thus allow water to work beneath it, a wash-out resulting. If the dam had other than a good solid end abutment, settlement at the arch ends might aid to raise the base at the crown of the arch ring. This is one of the reasons for using low units in the arch ring. The low unit will mean greater thickness of the latter and greater weight and stability for the dam.

Consideration of under-pressure, also, ought to lead to a design in which the batter is on the up-stream face, for such a batter adds to the pressure on the foundation by the amount of the weight of the water lying vertically over the battered face. Furthermore, a batter on the up-stream face tends to throw the end thrust of the arch downward, instead of in a horizontal direction, thus adding to the stability of the arch. Mr. Parker has made the greater part of the batter on the down-stream face. In the tangent portion the batter should be made on both faces, in order to obtain the spreading effect at the abutments, but, the writer is of the opinion that the stability would be greater if all the batter were made on the up-stream face in the arch portion.

Another reason for reducing the unit stress of an arch of this length is expressed by the term "column action". The arch resembles a column of very considerable unsupported length, just as a hollow tube under external pressure resembles such a column. A tube under external pressure acts about as a column of a length (hinged-ended) equal to half the circumference. Therefore, the ring of an arch dam should be treated as a column, and its length is equal to that of the arch from abutment to abutment. Of course, support is offered by the base of the dam. This is similar to the support afforded to a wide flange of a girder by the stiffness at the root of the flange. The arch dam is not, as some have claimed, equivalent to a short column.

Just what reduction should be made in unit stresses for wide, thin flanges is not very well understood or codified by structural engineers. A rule used in the case of girder flanges is to limit the thickness to one-twelfth of the width of the outstanding leg. Girder flanges are generally otherwise stiffened at intervals, and their maximum stress occurs only throughout a small portion of their length. The dam should be designed on a much safer basis than this, because, first, concrete has not the toughness of steel; second, the full stress exists from end to end of the arch; third, mere support on the earth is not

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equivalent to the strength at the root of a girder flange angle; and fourth, there are no braces at intervals in the arch dam. Mr. Parker's dam has a ratio of height to base thickness of six or seven, due to his commendable selection of a low unit stress. If he had elected to adopt the high unit stresses favored by some designers, the writer is of the opinion that the result would have been hazardous.

Another reason for low unit stresses is the uncertainty regarding the distribution of the stress throughout the section. Shrinkage of the concrete, expansion due to moisture, or changes due to varying temperature, are sure to result in unequal distribution of compressive stress on the section of the dam. The great length of the dam would make these variations of greater magnitude in this than in a smaller structure.

Hollow dams have proved most forcibly, by four notable failures, that the type having a bottom slab resting on the soil is an error. Three of these dams failed in winter, when, presumably, the weep-holes were frozen up; in the other case the weep-holes had been plugged up in order to save water. Upward pressure caused the failure of these hollow dams. It does not act to overturn them, but merely lifts them from the soil and thus permits the passage of water beneath the dam, which washes away the foundation, and failure results. Part of Mr. Parker's dam is hollow, but, fortunately, there is no bottom slab joining with the inclined slab, and the solid portion has considerable weight. It is probable that the rock formation is such as to preclude the possibility of water pressure under the solid portion being communicated across the gap between the inclined slab and the solid bottom part of the dam.

If this separation of the inclined slab and the solid bottom is intentional and general (and not merely accidental in the section, Fig. 3), it is fortunate, as it differentiates this dam from the hollow type which has furnished the four failures referred to.*

There is also considerable mass in the hollow portion of Mr. Parker's dam, and that helps to hold it down against upward pressure. Mr. Parker has used dowels to anchor the concrete to the masonry. Such dowels might help, but the writer would not be inclined to place much confidence in them, either for this purpose or for preventing the slipping of the arch abutments, where the line of thrust makes a rather sharp angle with the rock face. For the latter he would prefer to blast out the rock face in steps, the several steps being nearly normal to the line of thrust, or holes might be blasted for keys in order to prevent slipping. If he were building on a sloping rock, he would not wish to trust to dowels to prevent the structure from slipping down the slope, but would cut steps in the rock.

* Pittsfield Dam, *Engineering News*, April 1st, 1909; Dansville Dam, *Engineering Record*, January 1st, 1910; Stony River Dam, *Engineering News*, January 22d, 1914; and Plattsburg Dam, *Engineering News*, June 8th, 1916.

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In the north tangent the thrust is carried by rectangular columns or shafts between the piers. These members are reinforced with steel rods. The compression on these members is about twice as great as that on the arch ring. Twisted steel rods, 1 in. square, are placed longitudinally in these members, for the purpose of taking compression, and these rods are surrounded by what Mr. Parker calls hoops. It is unfortunate that the term "hoop" should be applied to a piece of steel bent in a rectangular shape. It is more unfortunate that it should be imagined that such pieces of steel can act in a manner similar to hoops in a hooped column. A mere term can be used in such a way as to create a false sense of security (witness the term "column" applied to two little channels tied together with small battens at intervals). A hooped column is a properly reinforced concrete member in compression, but, a member with rods disposed in a rectangle with square ties at intervals of about 12 in. is not a hooped column, and is not properly reinforced. The rectangular tie can only act to hold the four corner rods against lateral displacement, and the intervals between such ties should be closer than 12 in. in a properly hooped column.

Again, twisted rods are not of the proper shape for compression members; they should be as straight as possible.

Of course, the unit stress in these columns is not great, but, in the writer's judgment, it would be better to make the section of these members as large as that of the arch, and to use steel merely to tie the concrete together, and not to take compression.

The rodded column is the most discredited piece of construction that can be named. In almost all of more than a score of bad reinforced concrete wrecks rodded columns have proved their unfitness. Many large and chunky columns, carrying trifling loads, have broken into pieces, instead of standing and sustaining their share of the load.

Mr. Fortier.

Samuel Fortier,* M. Am. Soc. C. E. (by letter).—As the author has stated, the East Canyon Creek Dam forms part of a co-operative irrigation enterprise known as the Davis and Weber Counties Canal Company, which is owned and controlled by a community of farmers occupying the "Sand Ridge" immediately south of Ogden, Utah. For the past twenty-eight years, the writer has acted first as consulting engineer and, later, as adviser of this Company. The knowledge thus gained of the efforts made by its stockholders during the past quarter of a century to store water for irrigation purposes, is his only excuse for presenting in brief form a few ideas on two phases of this subject, *viz.:*

1.—The advisability—when financial and other conditions warrant —of designing and building dams to impound water for irrigation purposes in progressive stages.

* Berkeley, Cal.

2.—The advantages—where physical conditions permit—of building abutments of gravity section on each side of a canyon at the dam site, and confining the arch to the central portion of the structure. Mr. Fortier.

The first of these phases can best be considered in relation to the enterprise under discussion. In August, 1894, the writer made an examination of the dam site described by the author, and recommended that the company purchase the site and proceed to utilize it to the extent of the funds available and the credit then obtainable on the part of the company; and this was done. As rock was abundant and easily obtained, the writer recommended a loose rock dam having a steel core protected by asphalt concrete. The site selected by the writer lay well within the canyon, but, in excavating for a foundation, the officers of the company shifted the site near the upper entrance to the canyon, with a view to lessening the cost of building the foundation. Most of the troubles which subsequently affected this structure were directly traceable to this change of location. The original dam had an effective height of 58 ft. and formed a reservoir with a capacity of 3 850 acre-ft.

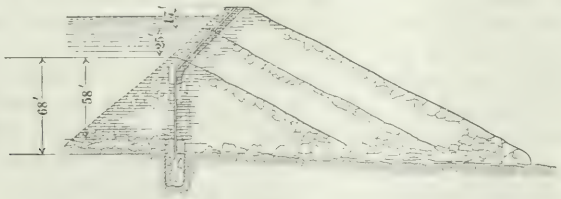

FIG. 7.

The topography of the site selected was well adapted to this comparatively low dam, but it was unsuited to the additions to the structure afterward imposed on the original. In 1900, the capacity of the reservoir was increased to 9 000 acre-ft. by raising the dam 25 ft. This increase could not be made by extending the steel core in a vertical direction, for the reason that the upper portion of the steel core as well as the upper portion of the rock-filling would have lain outside the canyon. (See Fig. 1.) Accordingly, the steel plates forming the core were inclined down stream at an angle of 45 degrees. Two years later, when the capacity of the reservoir was increased to 14 000 acre-ft. and the effective height of the dam to 100 ft., there seemed to be no other course than to place all the additional rock material on the down stream, as was done before, and continue the extension of the core-wall at the same inclination to the vertical. A cross-section of the original dam and the two additions is shown on Fig. 7.

In calling attention to the weakest feature of the old structure, it is only fair to add a word bearing on the success of the enterprise as a

Mr. Fortier.

whole. Before stored water from the reservoir was available, the farmers under the system were not in a prosperous condition. Being one of the late appropriators on the stream, the company was entitled to a portion of the annual flow of Weber River only when there was a surplus. Usually about July 1st of each year, when the discharge of the river was needed by prior appropriators, the company was compelled to close its head-gates and to keep them closed throughout the remainder of the crop-growing season. In consequence, only grain and partial crops of fodder could be raised. Neither orchard trees nor late vegetables could be grown with any assurance of success. With so limited and unreliable a water supply, the farmers could not produce the most profitable crops. At that time, the price of improved farming land under the system ranged from $10 to $30 per acre. Water stock of the company having a par value of $50 per share, was not worth more than $10 to $12, and a fair valuation of the property of the company would not have exceeded $250 000.

The use of water stored in the East Canyon Creek Reservoir gradually improved these conditions. As has been stated, the quantity stored was at first relatively small, but, in 1902, it had been increased to nearly 14 000 acre-ft. In 1910, the writer had occasion to examine the affairs of the company and found improved lands selling for $300 and more per acre, the value of the water stock had increased from $12 in 1896 to $175, and the value of the property of the company was conservatively estimated at $1 000 000. During the past eight years, the affairs of this company have improved at a more rapid rate than at any like period of its history, so that at present its financial condition is highly satisfactory.

This much of the history of this enterprise is cited in order to call attention to the feasibility and economic soundness of building dams to store water for irrigation purposes in progressive stages. Considered in the abstract and without reference to local conditions, most engineers would be inclined to condemn such a course as unsafe and unsound, and they might with reason undertake to prove the accuracy of their viewpoint by this same enterprise.

Here is a company that built a dam in 1896 at a cost of more than $50 000, enlarged it in 1900 at a cost of $35 000, enlarged it a second time in 1902 at a further cost of about $25 000, and, in 1915 and 1916, built a new structure from the foundation up at a cost of $193 000, thereby abandoning the greater part of the old structure. Thus, in a period of 20 years, $303 000, exclusive of interest charges, had been expended in storing water. At first sight, it would appear that good engineering practice as well as economy would have called for the construction in the first place of a permanent dam of ample capacity at a cost of less than $200 000.

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It is not difficult to show that such a course was impracticable, under conditions then existing, and even if it could have been done, of doubtful economy. The financial condition of the company in 1896 precluded the possibility of raising anything like $200 000 to store water. It was difficult to bond the property to raise $50 000 in order to provide funds to build the original dam. The interest on these bonds was 7½% per annum. Accordingly, the company would have been obliged to abandon the idea of storing water. Some other organization in all probability would have secured the valuable site on East Canyon Creek, and, to-day, the financial status of the Davis and Weber Counties Canal Company might have been in no better shape than it was prior to 1896.

For the sake of argument, let it be assumed that the stockholders of the company were in a position in 1896 to bond their property to construct a dam capable of impounding 28 000 acre-ft. In the light of subsequent developments, would this course have proved economically sound? The writer does not think so. With the possible exception of the period from 1896 to 1900, inclusive, the water users of the enterprise had sufficient water to meet their requirements and to provide for normal expansion. Unless the surplus could have been sold to other parties, the greater part of the 28 000 acre-ft. stored could not have been utilized. Furthermore, to base an estimate on what actually occurred, it is doubtful if much more than one-half of the storage of the large reservoir would have been utilized for the first 20 years after completion. In the meantime, the unused portion of the stored water would deposit its quota of silt within the reservoir and lessen to that extent its holding capacity.

Furthermore, during this 20-year period, the interest on the cost of a dam capable of impounding 28 000 acre-ft. in excess of the interest on the expenditures actually made in storing water in progressive stages, would have amounted to a large sum. The following figures may convey some idea of this extra interest charge on the basis of 7½%, compounded annually:

Interest on	$193 000	for	20	years	.....	$626 900
Interest on	50 000	"	"	"	.....	$162 410	
Interest on	35 000	"	16	"	.....	76 335	
Interest on	25 000	"	14	"	.....	43 815	282 560
					Balance		$344 340

This accounting of the two methods of building dams for the storage of water shows a balance of $344 340 in favor of the plan actually followed. Even when the rate of interest is reduced to 6% per annum,

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which corresponds more nearly to current rates for such undertakings, the balance in favor of the same plan would still be $230 170.

In order to be understood, it should be stated that the building of dams in separate stages or additions, as herein advocated, might not be economically sound if all the water stored by such structures could be utilized soon after completion. Such a course applies in particular to water stored for irrigation purposes, for the reason that long periods of time usually elapse before even one-half the volume of water stored is utilized. The profile, Fig. 8, based on the returns of the 13th Census, shows the average rate of settlement of irrigation enterprises in the United States.* The profile is based on data secured from 522 enterprises, each containing 5 000 or more irrigable acres. Generally

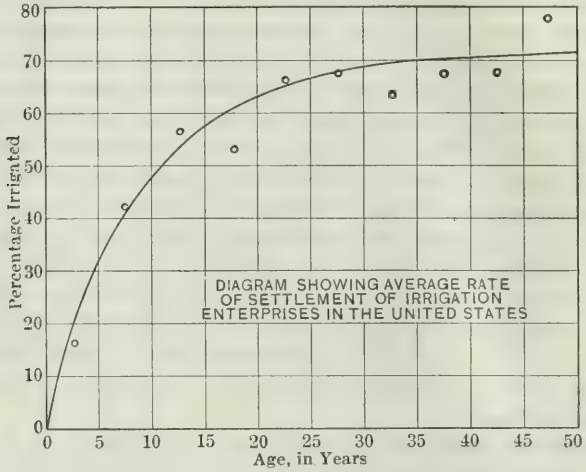

FIG. 8.

speaking, the larger the project, the slower the rate of settlement. Experience has shown that in projects containing 25 000 or more acres, a period of 20 years elapses on an average before 50% of the lands are settled.

In reference to the second phase, in which it is proposed to use a combination of two types of dams, the writer believes such a plan not only feasible but desirable where conditions are suitable.

The nearest approach to this combination type of concrete dam, in so far as the writer is aware, was first made use of by H. Hawgood, M. Am. Soc. C. E., in the design and construction of the Huacal Dam at Nacozari, Sonora, Mexico.† To quote Mr. Hawgood:

* "Irrigation in the United States," by R. P. Teele, D. Appleton Company, New York.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 564.

"The dam is not of the arch form for its entire length. For 42 ft. at its southeasterly end it is of gravity section, and in direction tangential to the curve." Mr. Fortier.

He also gives the reason for building a small portion of this structure in this manner, by stating:

"The continuance of the dam on a curve throughout its entire length would have brought the direction of its easterly end too closely parallel to the axis of the canyon to have been economical or safe."

When the writer was consulted in reference to the design of the East Canyon Creek Dam, he did not know that abutments of gravity sections had ever been used in connection with single arch dams. When the plan was proposed by Mr. Parker, the writer gave his approval and urged that both abutments be of this type, but owing to the formation of the rock at the north end, a hollow-deck section was substituted.

As a result of his observations of the behavior of the East Canyon Creek Dam during the past three years of operation, and his knowledge of dam sites throughout the West, the writer is of the opinion that a new type of combination dam might be developed, in which the abutments of the gravity sections at each end might well be enlarged in the case of spans longer than those of the Huacal Dam or the East Canyon Creek Dam.

A typical cross-section of a canyon at a dam site throughout the mountainous portion of the Far West, based on the observation of the writer, is shown in Fig. 9. It will be noted that the lower portion extending from bed-rock to nearly one-half the total height, is relatively narrow, with nearly vertical sides, and is filled more or less with sand, gravel, and boulders. From this point the sides slope in the form of the letter V as far as the sides of the canyon rise. Every engineer who has tried to design an arch dam that will fit into these conditions and be adapted equally as well to the narrow lower portion as to the flaring upper portion, has encountered difficulties. Maintaining a constant angle by varying the radii of the arch is only a partial solution, for, as the author has pointed out, cases are rare where it is possible to maintain a constant angle between the limiting radii.

The writer invites the attention of hydraulic engineers to the combination arch and gravity section type as one method of designing concrete dams to conform to cross-sections of sites such as are usually found in deep, rocky canyons. Referring to Fig. 9, the triangular portions, *ABE* and *CDF*, are to be designed as gravity sections, whereas the central portion is to constitute a single arch of substantially equal radii throughout its upper portion, if not throughout its entire height.

Some of the advantages possessed by the type of dam herein proposed are based on the following facts:

1. The central portion of gravity dams, comprising from one-quarter to one-third the total span, is by far the costliest to

Mr. Fortier.

build, owing to the height and width of base, which require a relatively large quantity of material.

2. By introducing an arch instead of a gravity section in the costliest portion of the dam, the compressive strength of the concrete rather than its weight is utilized, thus increasing the strength of the structure and lessening its cost.

Beside the saving in cost, this combination type of dam possesses, it is believed, other advantages, of which the expansion and contraction joint is one. It is now quite generally conceded that concrete requires such joints, and concrete dams are no exception to this rule. In the type under consideration, more than ordinary facilities are afforded to make ample provision for both expansion and contraction where the two types come together. Where the arch span is of small or medium

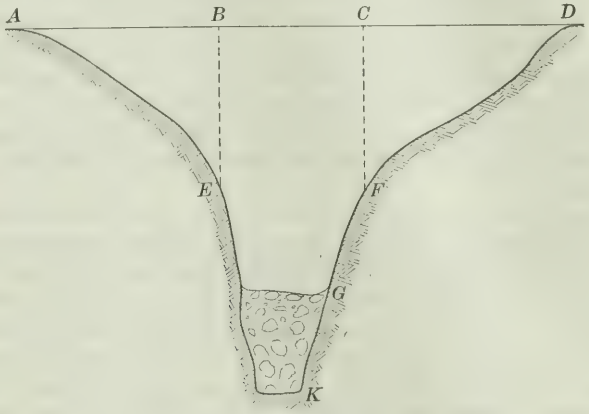

FIG. 9.

length, a joint at each end together with the usual steel reinforcing and the deflection of the arch under pressure should be ample to prevent cracks as well as harmful strains. The experience gained in observing such joints in the East Canyon Creek Dam leads to the conclusion that they are much more effective than those commonly used in all-gravity dams.

Another advantage of this type is its adaptability to the plan of building dams for irrigation purposes in two or more stages, as advocated previously in this discussion. Referring again to Fig. 9, it is readily seen that the foundation and that part of the superstructure up to the height, *F*, might well constitute the first stage. Subsequently, one of the abutments, in whole or in part, followed in time by the other, might be built. Lastly, the construction of the central arch would complete the undertaking.

Mr. Fortier.

In conclusion, the writer would call attention to the popularity of arched dams in recent years, notwithstanding the fact that relatively few dam sites are adapted to this type. In the foregoing, he has attempted to outline how dam sites in canyons which have not been properly formed by Nature for such purposes, can be built up in such a way as to afford ideal sites for single arch dams.

Mr. Wiley.

A. J. WILEY,* M. AM. SOC. C. E. (by letter).—The writer is particularly interested in this complete and satisfactory description of a simple arch dam of unusually great height, because of his experience with a dam of this type where the element of safety was not so important.

The Crowley Creek Dam is in Malheur County, Oregon, on a small stream, in a locality remote from settlement, where the only loss from failure would be that resulting to the owners who were willing to take any reasonable risk, in order to reduce the cost of the structure. The site was in a narrow canyon where an arch dam of short radius could be used. The foundations and abutments were of lava rock, and the only feasible location for the spillway was over the top of the dam. The design prepared by the writer was for a concrete arch dam, 60 ft. high above the stream bed, with a radius of 70 ft. The thickness on top and for 30 ft. below was 3 ft. At the base, 60 ft. below the crest, the thickness was 5.2 ft.

The Malheur Land and Livestock Company, for which the designs were prepared, had the dam staked out by an engineer, and employed a local contractor with some experience in concrete work to superintend the work, using the writer's plans and specifications, but without any further engineering aid. The work was carried to a height of 55 ft. above the stream bed in October, November, and December, 1907, and left at this height for a number of years.

The concrete was composed of Portland cement, sand, and gravel of a poorly graded lava character obtained from the creek bed, and the sand was rather fine and very dirty. About the only thing that could be said in favor of either sand or gravel was that they were the best that were available. Cement was used in the proportions of 1 bbl. to 1 yd. of concrete, and enough sand was used with the gravel to make a workable mix. Inspection after completion of the dam shows that little, if any, excavation was made in the rock base for the foundation, and the alignment was so seriously at fault that the radius at the top varied from a minimum of 58 ft. to a maximum of 80 ft.

The length of the dam on top was 170 ft. and, as no contraction joints were provided, a contraction crack opened at the middle point of the dam. This crack was about $\frac{3}{4}$ in. wide at the top of the dam

* Boise, Idaho.

Mr. Wiley. in cool weather, with the reservoir empty, and closed as the reservoir filled. The maximum overflow was about 2 ft. in depth and included large blocks of thick ice.

In September and October, 1915, under the supervision of Mr. A. T. Ingalls, of Boise, Idaho, an additional 15 ft. was placed on top of the dam, giving it a total height of 70 ft. above the stream bed or 10 ft. above that for which it was originally designed. It was intended that the dam should have no reinforcement, but in order to provide to some extent against temperature stresses, the engineer placed a 1-in. square bar at each 3 ft. of the 15-ft. increase in height.

It was found impossible to correct the poor alignment of the original work, so that the top of the completed dam has almost as great a variation in curvature and radius as that work.

Table 2 shows the thickness and the stresses, in tons per square foot, by the formula for thin arches, for various heights of dam for a radius of 70 ft. and for the depth of 2 ft. of overflow to which the dam has been subjected.

TABLE 2.

Depth below crest.	Thickness, in feet.	Stress in Concrete:	
		Tons per square foot.	Pounds per square inch.
0	3.0	1.45	20
10	3.0	8.75	122
20	3.0	16.00	224
30	3.0	23.30	324
40	3.0	30.60	425
50	3.6	31.60	440
60	4.4	30.80	430
70	5.2	30.30	420

The foregoing description is given to illustrate the abuses to which an arch dam sustaining high unit stresses may be subjected without failure.

The writer is also interested in the simple and efficient outlet works used at the East Canyon Creek Dam. This type of tower outlet controlled by three sets of cylindrical balanced valves, as described by the author, was, the writer believes, first used on the Oakley Dam, of the Twin Falls Oakley Land and Water Company, near Oakley, Idaho, and was designed by the writer in co-operation with J. L. Savage, M. Am. Soc. C. E. These outlet works were fully described in the *Engineering Record,* of January 11th, 1913, and have been in operation for about 5 years. They provide the outlet for a reservoir with a capacity of 70 000 acre-ft., and discharge a maximum of 500 sec-ft. The outlet is a tunnel through the rock at one end of the dam, which

Mr. Wiley.

is of the earth-fill type, 142 ft. high. The tunnel is provided with sluice-gates at its upper end, which are used for drawing off the small quantity of water in the lower 50 ft. The sluice-gates are seldom used, the cylindrical gates being used almost exclusively because of their much greater ease in operation. The cylindrical gates are set at intervals of 30, 60, and 90 ft. below the water surface in a tower 100 ft. high, continued by a shaft, 50 ft. deep, to an intersection with the outlet tunnel, and 20 ft. deeper to provide a water cushion at the bottom of the shaft.

The principal feature of the outlet works of the Oakley Dam are the balanced cylindrical valves which are described very clearly by the author. These valves have given excellent service, and the writer is glad to learn that they have been as successful under the maximum head of 150 ft. at the East Canyon Creek Dam as under the head of 90 ft. at the Oakley Dam.

The building of the outlet tower inside of a shaft as a protection against ice is an attractive feature of the author's design and may be necessary in that severe climate, especially if the reservoir contains much water in the winter season. The Oakley Reservoir is usually quite low in winter, and no ill effects have been experienced from the heavy ice, although the gate-tower is in the open reservoir without protection of any kind.

The writer notes that no attempt was made to shape the abutments of the East Canyon Creek Dam to radial lines, but thinks the statement that "the line of the rock made an angle with the line of the resultant of the arch thrust equal to the full angle of repose for masonry" should be read "the normal to the line of rock", etc. It is the writer's practice to specify that the abutments for an arched dam must be stepped back on roughly radial and tangential lines, and that the average line of the points of the steps must not make an angle with the radial line greater than 30 degrees.

The frost conditions under which the concrete was placed, as described by the author, were much more severe than the writer has been accustomed to, and the placing of concrete on a frozen surface, trusting to the removal of the frost by the warmth of the new concrete, is a practice that could only be resorted to under exceptional circumstances.

The cost of the concrete, given by the author as $6.37 per cu. yd., appears to include only the hauling of the cement and the mixing and placing of the concrete, and apparently omits the cost of the cement. If this item is included, the total cost would be $8.60 per cu. yd., a very low cost for reinforced concrete for the small volume and the difficult conditions under which this work was carried out. The writer would like to ask if the costs given are the actual costs or only

Mr. Wiley. contract prices, and, if the latter, whether the relation between contract price and actual cost is known?

Mr. Bellamy. HERBERT E. BELLAMY,* M. AM. SOC. C. E. (by letter).—This paper is of the greatest interest to the writer, particularly the two points describing (1) the contraction joints, and (2) the fine clayey sand used with the concrete. It must be gratifying to the author to know that the structure is a pronounced success. In the present age, engineers all the world over are so in the habit of reading of successful works accomplished, that it is a very rare thing, indeed, to hear of a defective or faulty one. However, it is the writer's intention to refer to a dam recently constructed, which is not entirely successful, due principally to (1) faulty contraction joints, and (2) faulty sand used in the concrete. The dam in question is known as the Ridgeway Storage Dam, and is used in connection with the city water supply of Hobart, Tasmania. This dam was completed in January, 1918, and was filled to top water level in September, 1918. Figs. 10 and 11, respectively, show the plan and elevation of the dam, and the reduced levels marked thereon are, in all cases, the heights above sea level. Fig. 12 is a view of the face of the dam from the east side.

The following are the principal dimensions of the wall:

Dimensions of Dam:

Top length of west wing	293 ft. 7 in.
" " " arch	232 " 9 "
" " " east wing	203 " 6 "

Maximum Depths:

Top of arch to foundation	201 ft.
" " " " bottom of shaft	285 "
" " " " east of reservoir	100 "
East wing, maximum depth	105 "
West wing, maximum depth	103 "

Thickness:

For whole length of wall	8 ft. at top
" " " " "	6 " below coping
Arch at bottom	46 ft. 6 in.
East wing at bottom	64 ft.
West wing at bottom	58 "

Dimensions of the Reservoir:

Area of water when full	19.3 acres
Distance round edge of water	64 chains (4 224 ft.)
Maximum depth of water	95 ft.
Capacity	207 000 000 gal.

* Hobart, Tasmania.

Mr. Bellamy.

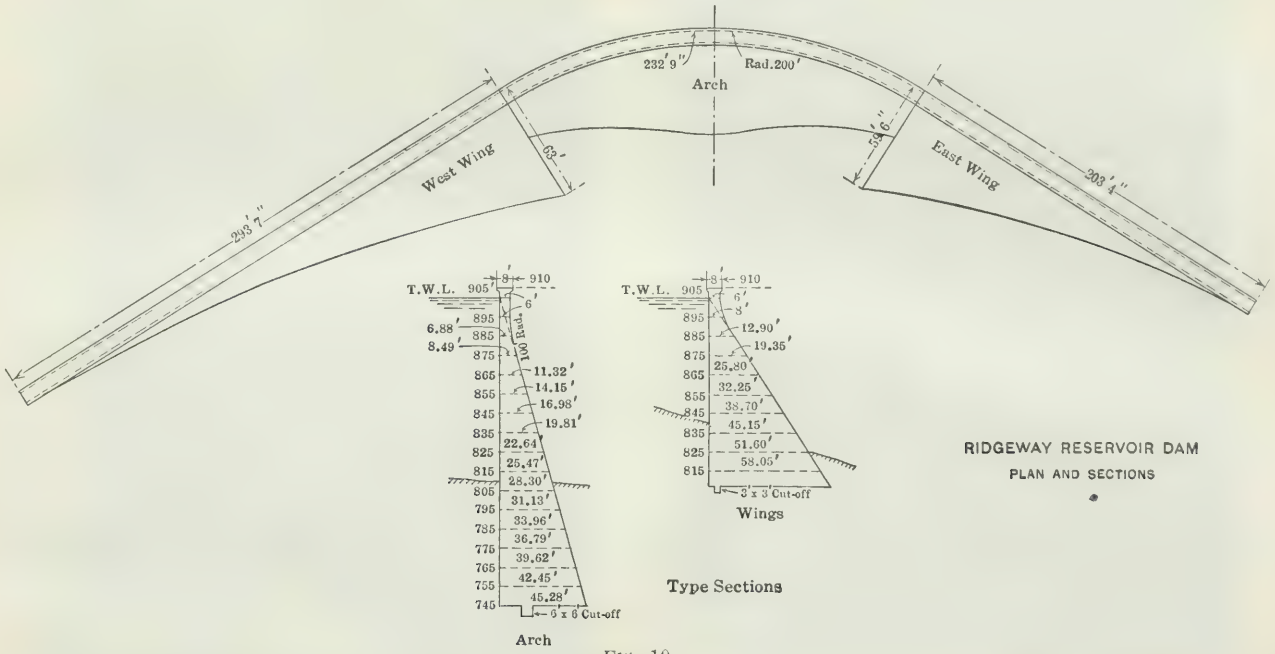

FIG. 10.

Mr. Bellamy.

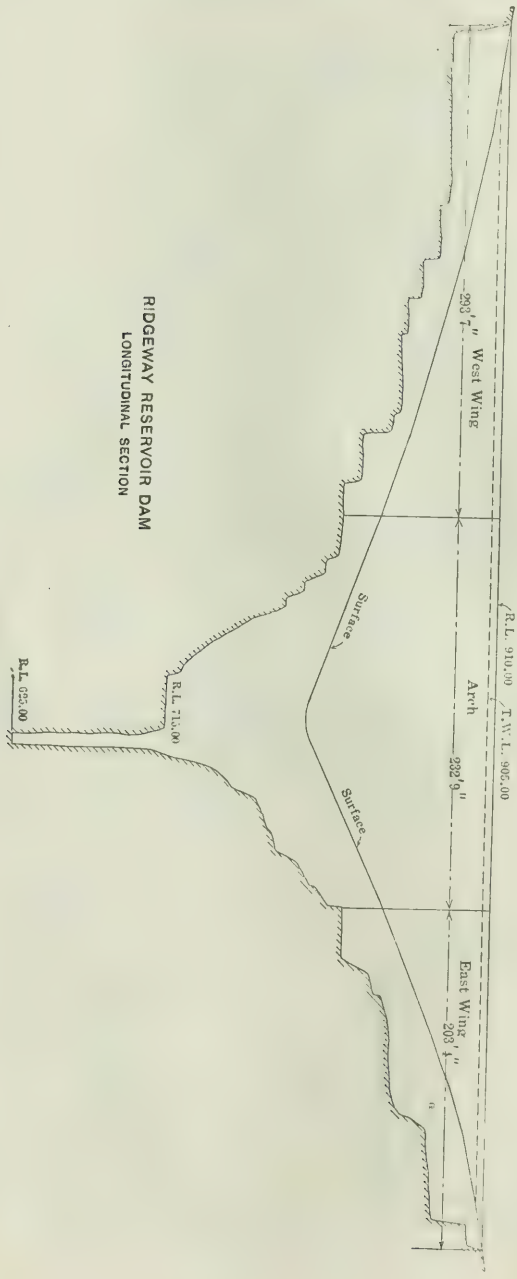

FIG. 11.

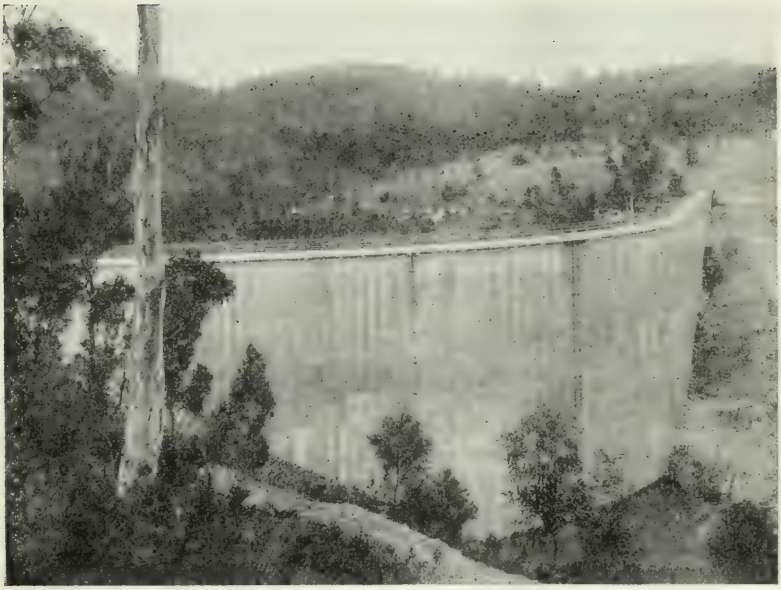

Fig. 12.—Face of Ridgeway Dam from East Side.

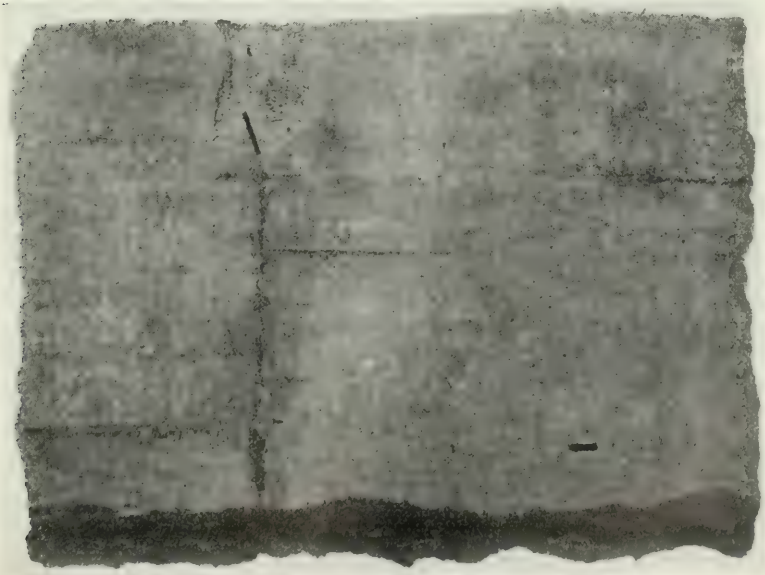

Fig. 13.—Parting Joint, Western Side, Ridgeway Dam.

Mr. Bellamy.

The wall consists of two gravity wings with an arch between them. The wings and arch are not connected. The wall is of concrete, in varying proportions, composed of diabase (which was found on the site) and crushed sandstone. The arch is reinforced with rails, vertically and horizontally, and a reinforcement of four rails is run from end to end of the dam near the top.

The outlet works consist of a concrete tunnel more than 1 000 ft. long, and a valve tower, 108 ft. high, which is approached by a small, three-span, reinforced concrete bridge.

Between the wings and the arch of the dam wall, a 6 by 6-in. bitumen joint with sheet lead faces was carried from the ground level to a distance of 2 ft. from the top of the wall, with the object of forming a water-tight key, as shown in Fig. 14. This method, however, was absolutely ineffective, and leakages took place through these joints as the reservoir was being filled. When a depth of 30 ft. of water in the reservoir was reached, there was a great leakage through the radial joint at the junction of the arch with the eastern gravity wall, but this stopped entirely as soon as the water level had reached a height of 40 ft. No further leakage has taken place through this joint.

Regarding the joint at the junction of the arch with the western gravity wall, a leakage took place as soon as there was 30 ft. of water in the dam, and this continued to increase in intensity until the reservoir was filled to top water level 905, that is, a depth of 95 ft. The estimated quantity of water leaking through this joint was no less than 250 000 gal. per day. As this leakage was considered to be excessive, and there was apparently no hope of it taking up, the writer recommended grouting the dam under pressure, but this advice was not adopted.

Reverting to the valve tower: It was found that water commenced to leak through the wall in several places when a depth of 30 ft. was reached in the reservoir, and the leakage continued right up the wall, increasing in volume until the top water level was reached. This leakage was considered to be even more serious than that through the dam wall, and was estimated to be 300 000 gal. per day. As this leakage could not be allowed to continue, the writer recommended that the reservoir be emptied, and that an entirely new internal lining, of special concrete, 6 in. thick, be placed from the bottom to the top of the tower. This work has now been completed with entirely satisfactory results. While the reservoir was being emptied in order to effect the necessary repairs to the valve tower, very careful examinations were made by the writer of the dam wall on the up-stream side. It was found that almost the whole area of the concrete face of the wall for a depth of some 6 in. had softened and could be easily broken off. Particular attention was paid to the concrete at the parting joint on the western side. When the water level was 40 ft. from the top of the

Mr. Bellamy.

dam wall, the photograph shown in Fig 13 was taken from a boat and indicates very clearly the parting joint on the western side. The concrete at the lower horizontal crack was absolutely rotten for some 2 ft. into the wall, and it was at this point that the major leak had occurred. In order to remedy the defects, the writer had a chase cut into the wall for the full depth downward, about 18 in. wide by 12 in. deep, and this was filled with especially made concrete. In addition, he proposes to render the whole face of the dam under pressure with the cement gun.

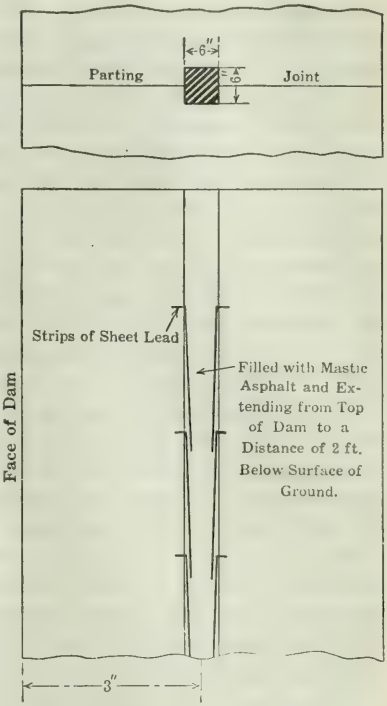
6"
Parting
Joint
Strips of Sheet Lead
Filled with Mastic Asphalt and Extending from Top of Dam to a Distance of 2 ft. Below Surface of Ground.
Face of Dam
3"

FIG. 14.

The question now leads one to consider the reason for the softening of the concrete. All the cement used throughout the works was of first-class quality and had passed all tests. The stone used was all that could be desired, but when the writer tested the disintegrated concrete, there was present a very large percentage of magnesium salts. The sand used in the concrete for the dam was obtained from a quarry almost adjoining the dam. This sandstone was taken from a small faulted block which had been carried up by the intruded greenstone, or diabase, a sill of which crowns the surrounding hills and in a hollow of which the reservoir has been constructed. A stratum of shale passes right through the sandstone which has been metamorphosed by the surrounding diabase and become impregnated by extraneous salts from the overlying decomposing rock. The writer also found several varieties of sandstone in the quarry. Analysis of the sand showed a large percentage of magnesium salts. Hence, the whole matter can be well explained as to why the concrete on the face of the wall and in the valve tower was unsatisfactory. The remedial measures which have thus been necessitated by the use of bad sand should serve to prove the absolute necessity of most thorough and searching investigations into all classes of sand likely to be used for structures; also, that short-time tests of sand and cement briquettes are not to be depended on.

A. F. Parker,* M. Am. Soc. C. E. (by letter).—According to reports arch dams have been built with unit stresses varying from 61 tons per sq. ft., down to 10 tons per sq. ft. They have also been built of poor materials and have departed considerably from true arch form; but, notwithstanding, there is no record of failure of an arch under water pressure. No one knows how great a loading from water pressure is required to break an arch, and, therefore, it is not known where the danger point lies or how near to it designs come. Mr. Parker.

All calculations to ascertain the strength of arch dams are based on theory and individual assumptions which vary according to the ideas of the designer. In other structures, the breaking point is often known, and thus a basis is established for intelligently applying a factor of safety, but, in all cases, facts, demonstrated by actual examples under use, finally govern. Why not frankly admit that there is no accurate knowledge of stresses in hydrostatic arches and adopt the simple formula for compressive strength in cylinders? Authorities do not agree as to the action of any stress except compressive. It will be found that, for arches not exceeding a compressive stress of, say, 25 tons per sq. ft., none of the requirements imposed by the use of numerous complicated formulas, all based on assumptions, calls for any increase of dimensions set by the simple formula.

Why not recognize precedents established by examples and adopt a factor of safety based on the actual compressive strength of the concrete used? This would be more rational than to rely on calculations based on unknown conditions which must be guessed at. From the published discussions of the subject, one can only conclude that designers have been proportioning arch dams by individual judgments; or, that is what it amounts to in the end.

There is only one rational way to ascertain, in advance of construction, the strength of concrete, and that is to fabricate actual concrete such as it is intended to use, mould it into test specimens of not less than 10 in. in diameter, so that the specimen is as truly like the concrete that will be produced on the job as it is possible to have it. Such test blocks broken in a compression machine of not less than 400 000 lb. capacity, at 30-day intervals until 6 months have elapsed, will give a basis on which to use a factor of safety.

The writer is fully convinced that the East Canyon Creek Dam is over-conservative with its unit stress of 12.5 tons per sq. ft. A procedure like the one stated above would have resulted in a unit stress of, at least, 25 tons. It is the writer's opinion that such a stress could have been safely used in designing this dam, with a corresponding saving in concrete.

So long as the rock on which the arch is built is as good as the concrete composing the arch, there is nothing to fear from water

* Ogden, Utah.

Mr. Parker.

pressure but failure under compressive stress. There is nothing to fear in an arch from uplift or pressure caused by water finding access to joints, just as long as the concrete weighs more than the water. The hydrostatic pressure is completely balanced on and resisted by the arch; this produces a definite compressive stress in the concrete of the arch.

This definite and balanced loading is wherein the arch dam differs from other arches and structures. There is no varying or live load; so it must be that calculations have a safer basis than in most cases. Ice thrust may be guarded against by vertical reinforcement, simply to distribute the ice pressure over more of the arch, above and below the ice.

The foregoing is prompted by reading Mr. Godfrey's discussion and will suffice to indicate the writer's opinion of it.

Mr. Fortier's discussion brings out some very interesting points. The development of the idea for building a dam by successive stages is well done; it must be remembered that it is in its adaptability to such practice that the arch differs from other forms of dams. The idea of using tangent gravity structures at the ends of the arch for it to abut upon where the natural contour of the canyon does not admit of using a full arch or, in other words, when the site does not admit of the use of an entire arch, to form an artificial canyon of the proper profile for an arch by means of these gravity sectioned tangent ends, is well illustrated in Mr. Bellamy's discussion. It is to be regretted that Mr. Fortier did not discuss the technical features of the dam.

Mr. Wiley's discussion gives a very interesting and instructive example of the superior qualities of arch dams. It is safe to say that in no other form than an arch can materials be made to withstand such abuse as is described in the case of the Crowley Creek Dam. It is notable that the unit stress reaches a maximum of 440 lb. per sq. in., or more than 31 tons per sq. ft. In spite of its imperfections and heavy unit stress, it withstood severe conditions of ice and floods. It adds important and convincing testimony to the increasing number of examples of abuse of workmanship and heavy loading that arch dams can stand up under.

In Mr. Wiley's comments on shaping the abutments to radial lines he states a sound principle, but, in this case, the work was carried out in a different way because of peculiar conditions that may not occur again. The design was made contemplating radial lines for all such joints and contact points.

It was an oversight that the cost of cement was not included in the cost of concrete. The figures were made up from the contractor's final estimate which did not include the cement and so it was overlooked. The prices are all contract prices, and the actual cost to the contractor of the whole or any item is not known.

Mr. Parker.

Mr. Bellamy's discussion shows an excellent example of using an arch for a dam in a canyon too wide to be filled entirely by the arch. It is an application of the principle so well stated by Mr. Fortier and shows, at a glance, the economy of cutting out the center and most costly portion of a gravity dam and substituting the cheaper arch form, cheaper because of less material in the structure and much cheaper by reason of reduced excavation for the foundation. Calculating by the simple formula a section 100 ft. below the top, it is found that the unit stress from compression is more than 22 tons per sq. ft. Also, the concrete materials are said to be poor; but still the dam does not fail. It is another argument for the idea that engineers are inclined to be too conservative in building arch dams.

Mr. Bellamy's whole discussion points to the wisdom, in designing an arch dam, of following the method of testing materials previously pointed out. Tensile tests of briquettes of cement and standard sand are only tests of the cement to show how it will behave with sand in making a mortar. The same kind of test of cement mortar made with other than standard testing sand is only a mortar test, and tensile at that. None of such tests is more than a rough, very rough, indication of what may be expected for compressive strength. The only true test is one made on large test blocks of the actual sizes and proportions of the aggregates used in the concrete.

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Paper No. 1438

ON A NEW PRINCIPLE IN THE THEORY OF STRUCTURES

By George F. Swain, Past-President, Am. Soc. C. E.

With Discussion by Messrs. C. W. Hudson, Edward Godfrey, Irving P. Church, John I. Parcel, L. J. Mensch, F. E. Turneaure, G. A. Maney, William H. Burr, A. C. Janni, H. T. Eddy, William Cain, and George F. Swain.

Synopsis.

This paper explains what is believed to be a new principle in the theory of structures, which has interesting and important applications. It is known that there is little new under the sun, and it may be that this principle has been known, and published elsewhere, but the writer has not seen it, and has arrived at it independently, from his own reflections on the subject.*

1.—Deflection of a Point.

The deflection of any point, A, of any structure may be found by a well-known formula, first published in America by the writer in 1883.† This formula is

$$\Delta = \sum \frac{s\,t\,l}{E\,A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)‡$$

* In Mohr's "Abhandlungen aus dem Gebiete der Technischen Mechanik." p. 425, this principle is hinted at, but it is not fully explained or its general applicability shown. The same thing is referred to in Grimm's book on Secondary Stresses, but incompletely elucidated.

† In a paper, entitled "On the Application of the Principle of Virtual Velocities to the Determination of the Deflection and Stresses of Frames," in the *Journal* of the Franklin Institute.

‡ If this expression causes confusion in the mind of the reader by reason of the fact that the left member is a length while the right member is apparently a force times a length, it is only necessary to remind him that in reality the left member is a length times a unit force; or, differently stated, since t is the stress due to a unit force, it may be considered as a stress due to any given force divided by that force, and, therefore, is a ratio. At any rate, the equation as it stands is perfectly correct.

in which, t = stress in any member due solely to a load of unity acting at the point the deflection of which is desired, under the given actual loads, in the direction in which said deflection is desired;

s = stress in the same member, due solely to the given actual loads;

l = length of the same member;

A = area of the same member;

E = modulus of elasticity of the same member.

This formula, as is well known, may be applied, not only to finding the deflection of a joint of a framed structure, but also to finding the deflection of any point of a beam. In the latter case, the area, A, must be replaced by an infinitesimal area, da, and the stresses, s and t, must be the stresses on such area. If the case is one of pure flexure, with no resultant axial force on any section, and if M_t is the bending moment due solely to the force unity, and M_s is the bending moment due to the actual loads, at any given section, the formula becomes

$$\Delta = \int \frac{M_s \, M_t \, d \, l}{E \, I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

the integral being extended over the entire length.

The principle now to be explained is a similar one which affords a means of finding the rotation of the axis of any piece at any point.

2.—Angular Rotation of the Axis of a Piece at a Point. (The New Principle).

In addition to finding the deflection of any point of a structure in any given direction, it is often desired to find the angular rotation at a point, *i. e.*, the change of direction of the axis of a piece. This may be obtained by a general method analogous to that used for finding the deflection.

Let it be desired to find the angular rotation, or change of direction of the axis of a piece, at a given point, A, due to any given loads. At the given point apply only a couple the moment, M, of which equals unity, and let the corresponding angular rotation, α, at A be first determined by the method of work. (It may be thought a contradiction, or an impossibility, to apply a couple at a point, but this obviously is the same as applying two very large forces a very short distance apart,

or two forces each equal to $\frac{1}{d\,x}$ at a distance, $d\,x$, apart. This is perfectly conceivable.) The work done by the couple the moment of which is $M = 1$, if gradually applied, will obviously be $\frac{\alpha}{2}$, since the couple may be considered to consist of forces of unity a unit distance apart, and its average value is $\frac{1}{2}$. The only outer forces being this couple, the work of these outer forces is $\frac{\alpha}{2}$, and this must equal the work of the inner forces. On any bar or fiber having a uniform stress intensity throughout its area and length, due to the applied moment, $M = 1$, let r equal the total direct stress on the bar or fiber and $\Delta\, l$ its change of length. Then the equation between the work of the outer forces and that of the inner forces is

$$\frac{\alpha}{2} = \sum \frac{r}{2} \text{ (change of length), or } \alpha = \Sigma\, r \text{ (change of length).}$$

Now, as in the case of deflection, it is clear that the deformation is a purely geometrical phenomenon, and that any change of length of a bar or fiber, whatever its cause, will produce the same angular rotation at A. Hence, if for "change of length" in the above formula, which is the change of length due to $M = 1$ at A, we substitute the change of length due to any given loading, the formula will give the angular rotation at A due to the given loading, or

$$\alpha = \sum \frac{r\,s\,l}{E\,A} \qquad (3)$$

in which, for any given bar or fiber,

E = modulus of elasticity;
A = area;
l = length;
r = stress on the area, A, due to the couple, $M = 1$, at the given point in a given sense;
s = stress on the area, A, due to the applied loads.

The applied couple, M, may be applied with either a right-handed or left-handed direction of rotation. If α comes positive, this means that the given loads produce a rotation in the same direction as that assumed for M.

In using this equation, tension and compression, of course, should be distinguished in sign, carefully using one of these terms consistently as positive and the other as negative.

If the stresses include bending stresses, the elementary fibers, *da* in area and *dl* in length, must be taken.

The formula shows the effect of direct stresses only, shearing effects being neglected.

In applying the couple, $M = 1$, bending stresses may be produced in a bar, in addition to a direct stress in the same bar. The applied loads may produce bending or direct stresses, or both, in any bar. In such cases, for any fiber, s and r are the stresses on an area, *da*.

In general, if the applied loads cause in any bar at any section a direct force, T_s, and a flexural moment, M_s, the value of s will be

$$s = \left(\frac{T_s}{A} + \frac{M_s y}{I}\right) d a$$

and if the couple, $M = 1$, produces in the same bar and section a direct force, T_r, and a moment, M_r, the value of r will be

$$r = \left(\frac{T_r}{A} + \frac{M_r y}{I}\right) d a.$$

The value of the differential, $d\alpha$, therefore, will be

$$d\alpha = \left(\frac{T_s\, d a}{A} + \frac{M_s y\, d a}{I}\right)\left(\frac{T_r d a}{A} + \frac{M_r y d a}{I}\right)\frac{d l}{E d a}$$

$$= \frac{1}{E}\left[\frac{T_s T_r d a . d l}{A^2} + \frac{T_s M_r y . d a . d l}{A I} + \frac{T_r M_s y . d a . d l}{A I} + \frac{M_s M_r y^2 . d a . d l}{I^2}\right]$$

It is supposed that every bar is straight, with one principal axis of each cross-section in the plane of the applied loads, in which plane also the couple, $M = 1$, is applied. I is the moment of inertia about the principal axis perpendicular to that plane, which, of course, passes through the center of gravity of the section, so that $\int y\, d a = 0$.

This differential refers to an infinitesimal length, *dl*, of any fiber of area, *da*, in any bar, and must be integrated over the cross-section, then over the length of the bar, and the sum of the results taken. The final result will easily be seen to be, if A is constant throughout the length of each bar,

$$\alpha = \Sigma \frac{T_s T_r l}{E A} + \Sigma \int_0^l \frac{M_s M_r d l}{E I} \quad \ldots\ldots\ldots\ldots (4)$$

The forces, T_r and T_s, are the direct forces acting in any bar, and are supposed to be constant throughout its length; and the moments, M_r and M_s, are the bending moments at any section of any bar. Each of the forces, s and r, in Equation (3) may, in any bar, represent a direct stress only, or flexural stress only, or both. The following results clearly follow from Equation (4):

1.—If, in any bar, either the actual load system, or the applied moment, $M = 1$, causes a direct stress only, and the other a flexural stress only, that bar may be neglected in the computation; for the value of α for that bar is zero.

2.—The flexure in any bar may be neglected unless both the actual loads and the applied moment, $M = 1$, cause flexural stress in it.

3.—The direct stress in any bar may be neglected unless both the actual loads and the applied moment, $M = 1$, cause a direct stress in it.

4.—If the case is one in which there is only flexural stress in the bars, without direct stress, the formula becomes

$$\alpha = \Sigma \int_0^l \frac{M_s M_r . dl}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

5.—If A as well as T varies along the length of any bar, the formula becomes

$$\alpha = \Sigma \int_0^l \frac{T_s T_r \, dl}{E A} + \Sigma \int_0^l \frac{M_s M_r . dl}{E I} \quad \ldots\ldots\ldots\ldots (6)$$

These principles apply in using Equation (1), to determine deflection, as in using Equation (3), to determine rotation.

3.—Illustrations of Angular Rotation.

Although this method would seldom or never be used to find the slope of a beam exposed to flexure, this case may be illustrated to show the applicability of the method.

Case I.—Slope of a Beam Fixed at One End.—Let it be desired to ascertain the slope of the free end of a beam of constant section (Fig. 1), fixed at one end and loaded at the other with a load, P. The load, P, produces on any section, distant x from the end, a moment, $M_s = -Px$. The applied couple, $M = 1$, at the free end, if acting in the sense shown, likewise produces

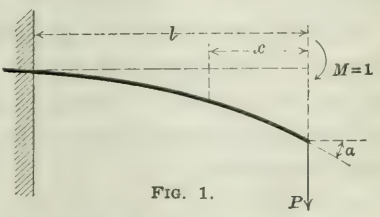


FIG. 1.

a moment at the same section, $M_r = -1$. There is no resultant direct stress on any cross-section. Hence, by Equation (4)

$$\alpha = \int_0^l \frac{Px\,dx}{EI} = \frac{Pl^2}{2EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which is the well-known value.

Again, let it be desired to find the slope at the end when a couple with moment $= M_0$ is applied at the end:

$$M_s = -M_0$$
$$M_r = -1$$
$$\text{therefore, } \alpha = \frac{M_0 l}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

This result is obtained with equal (but no greater) ease by the moment-area method.

Case II.—The Slope at the Ends of a Simple Beam Supported at Both Ends.—(*a*).—With a single concentrated load (Fig. 2):

Let it be desired to obtain the slope (α) at the right end of the beam of constant section shown in Fig. 2, which also shows the moment curve for M_s.

Then, clearly,

$M_s = \dfrac{Px(l-a)}{l}$ from left abutment to P,

$M_s = \dfrac{Pa(l-x)}{l}$ from right abutment to P,

$M_r = \dfrac{x}{l}$

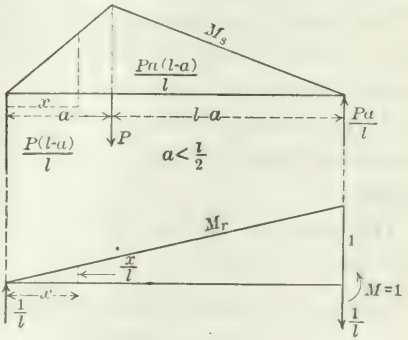


FIG. 2.

$$\text{Then } \alpha = \frac{1}{EI}\int_0^a \frac{P(l-a)}{l^2}\cdot x^2\,dx + \frac{1}{EI}\int_a^l P\,\frac{a}{l^2}(lx - x^2)\,dx$$
$$= \frac{Pa}{6EIl}\cdot(l^2 - a^2) \quad \ldots\ldots\ldots\ldots\ldots (9)$$

in the direction shown, that is downward to the left.

This is the simplest method of obtaining this result.

The slope at the load (α_l) may be obtained easily, by this method, by applying $M = 1$ at the load; thus (Fig. 3):

$M_s = \dfrac{P\,x\,(l-a)}{l}$ from left abutment to load,

$M_s = \dfrac{P\,a\,x_1}{l}$ from right abutment to load,

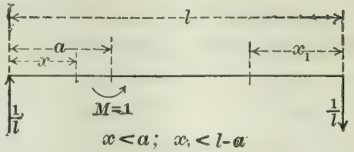


FIG. 3.

$M_r = \dfrac{x}{l}$ from left abutment to load,

$M_r = -\dfrac{x_1}{l}$ from right abutment to load,

$$\alpha_l = \frac{1}{E\,I}\int_0^a \frac{P\,x^2\,(l-a)\,dx}{l^2} - \frac{1}{E\,I}\int_0^{l-a} \frac{P\,a\,x_1^{\,2}\,dx}{l^2}$$

$$= \frac{P\,a\,(l-a)(2\,a-l)}{3\,E\,I\,l}$$

This is a negative quantity, showing that at the load the axis slopes in a contrary direction to that indicated by the applied moment, that is, it slopes downward to the right.

The point where the axis is horizontal may be easily found by this method. Let this point be distant Q_0 from the right end, that is, in the longer segment; and let a moment, $M = 1$, be applied there. The slope there being zero, find $\alpha_0 = 0$, as follows:

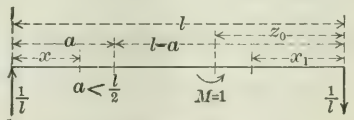


FIG. 4.

The values of M_s and M_r will be as follows (Fig. 4):

Between left end and load: $M_s = \dfrac{P\,x\,(l-a)}{l}$

$$M_r = \frac{x}{l}$$

Between right end and z_0: $M_s = \dfrac{P\,a\,x_1}{l}$

$$M_r = -\frac{x_1}{l}$$

Between z_0 and load: $M_s = \dfrac{P\,a\,x_1}{l}$

$$M_r = -\frac{x_1}{l} + 1$$

$$E\,I\,\alpha_0 = 0 = \int_0^a \frac{P\,x^2\,(l-a)}{l^2}\,dx - \int_0^{z_0} \frac{P\,a\,x_1^2}{l^2}\,dx_1 - \int_{z_0}^{l-a} \frac{P\,a\,x_1^2}{l^2}\,dx_1$$

$$+ \int_{z_0}^{l-a} \frac{P\,a\,x_1}{l}\,dx_1, \text{ or } z_0 = \sqrt{\frac{l^2 - a^2}{3}}$$

(*b*).—To find slope at the right end due to a moment, M_1, at the left end, causing compression in the top fibers (Fig. 5):

$$M_s = \frac{M_1\,(l-x)}{l}$$

$$M_r = \frac{x}{l}$$

$$\alpha = \int_0^l \frac{M_1\,x\,(l-x)\,dx}{l^2\,E\,I} = \frac{M_1\,l}{6\,E\,I} \quad \ldots\ldots\ldots\ldots (10)$$

acting in the direction of the moment, $M = 1$, or downward to the left.

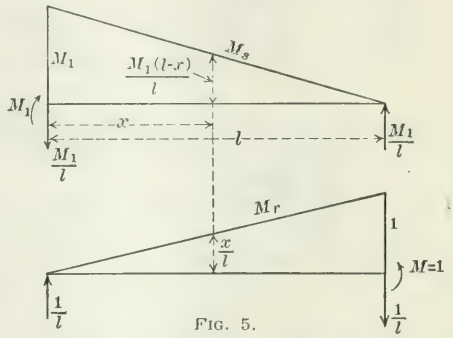


Fig. 5.

(*c*).—To find the slope at the right end due to a moment, M_2, at the right end, causing compression in the top fibers (Fig. 6):

$$M_s = \frac{M_2\,x}{l}$$

$M_r = \frac{x}{l}$, as before,

$$\alpha = \int_0^l \frac{M_2\,x^2\,dx}{l^2\,E\,I} = \frac{M_2\,l}{3\,E\,I} \quad \ldots (11)$$

downward to the left.

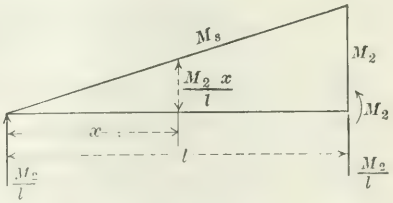


Fig. 6.

Case III.—The Continuous Girder.—Theorem of Three Moments.—This principle affords what the writer believes to be the simplest method of demonstrating the Theorem of Three Moments, which follows immediately from Case II. See Fig. 7.

Fig. 7 shows two consecutive spans of a continuous girder, l_{n-1} and l_n, each with a single concentrated load in it. The span, l_{n-1}, is exposed to the action of its load, and to moments, M_{n-1} and M_n, at the supports. In addition, the level of the support, n-1, may be lowered so that it is h_{n-1} above a base line, while the support, n, is h_n above that base. The

slope at n, in the direction downward to the left, resulting from P_{n-1}, M_{n-1}, M_n, and the change of level of supports will be, from Equations (9), (10), and (11):

$$\alpha_n = \frac{P_{n-1}\, a_{n-1}\,(l^2_{n-1} - a^2_{n-1})}{6\,E\,I\,l_{n-1}} + \frac{M_{n-1}\,l_{n-1}}{6\,E\,I} + \frac{M_n\,l_{n-1}}{3\,E\,I} + \frac{h_n - h_{n-1}}{l_{n-1}} \cdot (12)$$

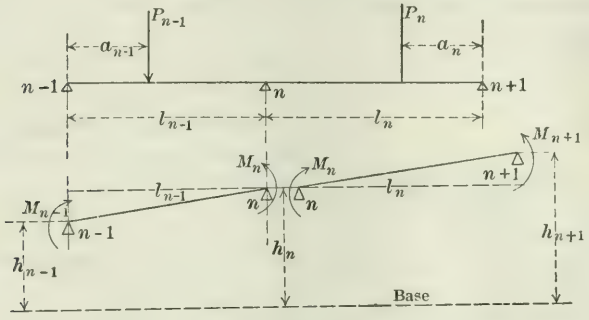


FIG. 7.

The span, l_n, is likewise exposed to the action of P_n, and of moments, M_n and M_{n+1}, and also the support, $n+1$, may be raised above n. Under these conditions, the slope at n, in a direction downward to the right, will be

$$\alpha'_n = \frac{P_n\, a_n\,(l_n^2 - a_n^2)}{6\,E\,I\,l_n} + \frac{M_{n+1}\,l_n}{6\,E\,I} + \frac{M_n\,l_n}{3\,E\,I} - \frac{h_{n+1} - h_n}{l_n} \ldots . (13)$$

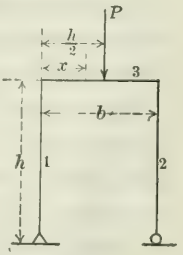
But, if the beam is continuous, $\alpha_n = -\alpha_n'$, or $\alpha_n + \alpha_n' = 0$; hence, for a series of loads, P_{n-1} and P_n,

$$M_{n-1}\,l_{n-1} + M_{n+1}\,l_n + 2\,M_n\,(l_{n-1} + l_n) = 6\,E\,I\left(\frac{h_{n+1} - h_n}{l_n} - \frac{h_n - h_{n-1}}{l_{n-1}}\right)$$

$$- \sum \frac{P_n\, a_n}{l_n}\,(l_n^2 - a_n^2) - \sum \frac{P_{n-1}\, a_{n-1}}{l_{n-1}}\,(l^2_{n-1} - a_{n-1})^2 \ldots (14)$$

which is the most general form of the Theorem of Three Moments.

Case IV.—Slope (α) at the Foot of One Post of a Rigid Rectangular Bent Supported on Pins, and One Pin on Rollers.—In Fig. 8, owing to the rollers, there are no horizontal reactions at the supports. By symmetry, each vertical reaction is $\frac{P}{2}$.



The applied load, P, produces, in each post, merely a compression, $\frac{P}{2}$; and, in the upper piece, flexure, with a triangular moment diagram having a center ordinate, $\frac{P\,b}{4}$.

The applied couple, $M = 1$, produces (Fig. 9) in the left post a compression of $\frac{1}{b}$; and in the right post a tension of $\frac{1}{b}$, together with a uniform bending moment of $M = 1$ at each point, and in the upper piece a bending moment causing compression in the top fibers, and varying uniformly from zero at the left support to 1 at the right support.

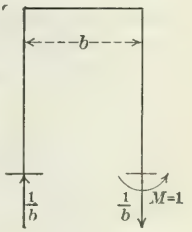


FIG. 9.

From Equation (2) we find, numbering the parts as shown,

$$\alpha = \frac{P\,h}{2\,b\,E_1\,A_1} - \frac{P\,h}{2\,b\,E_2\,A_2} + \int_0^{\frac{b}{2}} \frac{P\,x^2\,d\,x}{2\,b\,E_3\,I_3} + \int_{\frac{b}{2}}^{b} \frac{P}{2}\,(b - x)\,\frac{x\,d\,x}{b\,E_3\,I_3}$$

$$= \frac{P\,h}{2\,b\,E_1\,A_1} - \frac{P\,h}{2\,b\,E_2\,A_2} + \frac{P\,b^2}{16\,E_3\,I_3}$$

If, as would generally be the case, $E_1 = E_2 = E_3 = E$, and $A_1 = A_2$,

$$\alpha = \frac{P\,b^2}{16\,E\,I_3}$$

or the same as at the end of the upper beam loaded at the center, as it obviously should be.

Case V.—A King-Post Truss.—Let it be required (Fig. 10) to find the angular rotation (α) of the end of the top chord, all joints being supposed to be connected by frictionless pins. The load, P, is at the center.

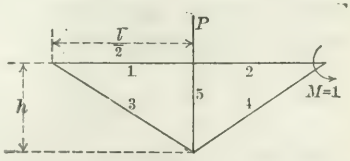


FIG. 10.

For the bars numbered, the values of l, r, and s, are given in Table 1.

The couple, $M = 1$, in the direction shown, causes an upward reaction of $\frac{2}{l}$ on Bar 2 at the left end, and an equal downward reaction at its right end, in other words, a compression on Bar 5 equal to $\frac{2}{l}$, and an upward force of $\frac{2}{l}$ on the end pin at the abutment. This com-

TABLE 1.

Bar.	Length.	Stress, r, due to $M = 1$ at right end of 2.	Stress, S, due to the load, P.
1	$\frac{l}{2}$	$-\frac{1}{2h}$	$-\frac{Pl}{4h}$
2	$\frac{l}{2}$	$-\frac{1}{2h}$ + flexure	$-\frac{Pl}{4h}$
3	$\sqrt{h^2+\frac{l^2}{4}}$	$\frac{1}{lh}\sqrt{h^2+\frac{l^2}{4}}$	$\frac{P}{2h}\sqrt{h^2+\frac{l^2}{4}}$
4	$\sqrt{h^2+\frac{l^2}{4}}$	$\frac{1}{lh}\sqrt{h^2+\frac{l^2}{4}}$	$\frac{P}{2h}\sqrt{h^2+\frac{l^2}{4}}$
5	h	$-\frac{2}{l}$	$-P$

pression on Bar 5 causes tension in Bars 3 and 4 and a downward pull on the pin at the right end of Bar 2 equal to $\frac{1}{l}$, leaving a resultant upward force of $\frac{1}{l}$ on the pin, which must be balanced by a downward reaction exerted by the abutment, as must clearly be the case to balance $M = 1$, independent of any internal stresses. Similarly, the upward reaction at the left abutment is $\frac{1}{l}$.

By the second principle, explained in Paragraph 2, the flexure in Bar 2 does not enter into the equation for slope; hence, the material being uniform, and each bar of constant section and uniform material, if $A_1 = A_2$, and $A_3 = A_4$,

$$E.\alpha = \frac{P\,l^2}{8\,h^2\,A_1} + \frac{P}{l\,h^2\,A_3}\left(\frac{l^2}{4}+h^2\right)^{\frac{3}{2}} + \frac{2\,P\,h}{l\,A_5} \quad \ldots\ldots(15)$$

This may be checked by the usual method, which would be to find first the deflection, Δ, of the top of 5, by Table 2.

$$\Delta = \sum \frac{s\,t\,l}{E\,A} = \frac{1}{E}\left[\frac{P\,l^3}{16\,h^2\,A_1} + \frac{P}{2\,h^2\,A_3}\left(\frac{l^2}{4}+h^2\right)^{\frac{3}{2}} + \frac{Ph}{A_5}\right]$$

$\alpha = \frac{2\,\Delta}{l}$, which is the same as Equation (15).

Case VI.—A Framed Bent with Columns Rigidly Fixed at the Bottom.—Consider the bent shown in Fig. 11, and suppose all joints hinged except where the knee-braces meet the columns. The structure

is then statically determined as regards the inner forces, but not as regards the outer forces. Suppose that it is imagined to be made determined by having each column supported on a frictionless pin, and one of them (the right-hand one) on a roller. Under these circumstances, the structure will be entirely statically determined, but the distance between the feet of the columns will change (increase generally), and the axis of the columns at the bottom will rotate. To bring the structure into the condition in which it actually is, apply a horizontal inward force, H_0, at the bottom of the right-hand column, a couple with left-handed moment, M_1, at the bottom of the right-hand column, and a couple with right-hand moment, M_2, at the bottom of the left-hand column. If these three quantities, H_0, M_1, and M_2, can be found, the problem is solved. The conditions which determine them are that the resultant change of span is zero, and that the resultant rotation of the bottom of each column is zero. It remains to formulate these conditions.

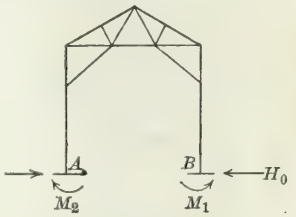


FIG. 11.

In any bar (or fiber, if there is flexure) of the statically determined structure, let

S_0 = stress due to the actual loads,
t = " " " an outward force of unity at B,
r_1 = " " " a right-handed moment of unity at B,
r_2 = " " " " left-handed " " " " A,

Then the actual stress in that bar or fiber will be

$$S = S_0 - H_0 t - M_1 r_1 - M_2 r_2$$

The outward deflection of B, which is zero, will be

$$\Delta = 0 = \sum \frac{s\,t\,l}{E\,A} = \sum \frac{s_0\,t\,l}{E\,A} - H_0 \sum \frac{t^2\,l}{E\,A} - M_1 \sum \frac{r_1\,t\,l}{E\,A} - M_2 \sum \frac{r_2\,t\,l}{E\,A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The rotation of B in a right-handed direction, which is zero, will be

$$\alpha_1 = 0 = \sum \frac{s\,r_1\,l}{E\,A} = \sum \frac{s_0\,r_1\,l}{E\,A} - H_0 \sum \frac{t\,r_1\,l}{E\,A} - M_1 \sum \frac{r_1^2\,l}{E\,A} - M_2 \sum \frac{r_1\,r_2\,l}{E\,A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

TABLE 2.

Bar.	Length.	Stress, t, due to load 1 at top of Bar 5.	Stress, s, due to load, P, at top of Bar 5.
1	$\frac{l}{2}$	$\frac{l}{4h}$	$-\frac{P}{4h}$
2	$\frac{l}{2}$	$-\frac{l}{4h}$	$\frac{Pl}{4h}$
3	$\sqrt{h^2+\frac{l^2}{4}}$	$\frac{1}{2h}\sqrt{h^2+\frac{l^2}{4}}$	$\frac{P}{2h}\sqrt{h^2+\frac{l^2}{4}}$
4	$\sqrt{h^2+\frac{l^2}{4}}$	$\frac{1}{2h}\sqrt{h^2+\frac{l^2}{4}}$	$\frac{P}{2h}\sqrt{h^2+\frac{l^2}{4}}$
5	h	-1	$-P$

The rotation of A in a left-handed direction, which is zero, will be

$$\alpha_2 = 0 = \sum \frac{s\, r_2\, l}{E\, A} = \sum \frac{s_0\, r_2\, l}{E\, A} - H_0 \sum \frac{t\, r_2\, l}{E\, A} - M_1 \sum \frac{r_1\, r_2\, l}{E\, A} - M_2 \sum \frac{r_2^2\, l}{E\, A} \quad \ldots\ldots(18)$$

If the material is uniform throughout, E may be cancelled.

All the stresses indicated in these equations may be found by statics, and tabulated, and the summations found. Designating the sums by letters, a, b, c, d, e, f, g, h, k, the equations may be written

$$a = H_0 b + M_1 c + M_2 d \quad \ldots\ldots(16)$$
$$e = H_0 c + M_1 f + M_2 g \quad \ldots\ldots(17)$$
$$h = H_0 d + M_1 g + M_2 k \quad \ldots\ldots(18)$$

in which all the constants are known, and the problem is to find H_0, M_1, and M_2.

The solution gives the following values:

$$M_1 = \frac{(a\,d - h\,b)(d\,c - b\,g) - (a\,c - e\,b)(d^2 - k\,b)}{(c\,d - g\,b)^2 - (c^2 - f\,b)(d^2 - k\,b)} \quad \ldots\ldots(19)$$

$$M_2 = \frac{(a\,c - e\,b)(d\,c - b\,g) - (a\,d - h\,b)(c^2 - f\,b)}{(c\,d - g\,b)^2 - (c^2 - f\,b)(d^2 - k\,b)} \quad \ldots\ldots(20)$$

$$H_0 = \frac{a - M_1\, c - M_2\, d}{b} \quad \ldots\ldots(21)$$

These three equations completely solve this case. In any actual case, the values in these equations will be numerical, and can easily be substituted. To obtain a general value of M_1, M_2, and H_0, in any special

case, by this method, is possible, but is long and complicated, and other methods are very likely to be shorter and more easily visualized.

In this, as in all statically undetermined cases, it is necessary, in designing, first to make some assumptions which will make the structure statically determined, design the sections on this assumption, and then use the foregoing equations for a more exact design, repeating the process if necessary.

In the present case, if the applied loads and the outline of the structure are symmetrical about a vertical center line, it is easily seen that $e = h$, $c = d$, $f = k$, and that $M_1 = M_2$, hence

$$M_1 = M_2 = \frac{ac - eb}{2c^2 - gh - fb} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

$$H_0 = \frac{2ec - a(g + f)}{2c^2 - b(g + f)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The point of inflection in either post, in this case, will be at a height above the base

$$h_0 = \frac{M_1}{H_0} = \frac{ac - eb}{2ec - a(g + f)}.$$

In obtaining any of the sums required, the principles relating to combinations of direct stress with flexure, explained in Paragraph 2, apply.

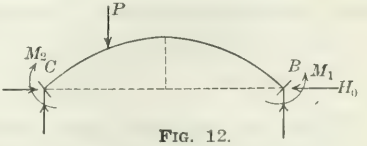


FIG. 12.

Case VII.—The Arch Without Hinges.—In this case (Fig. 12) there are three equations to be supplied by elastic conditions, namely:

(1).—The change of span = 0,
(2).— " " " slope at $B = 0$,
(3).— " " " " " $C = 0$.

Suppose, first, the arch to be statically determined as regards the outer forces, by being supported on frictionless pins, and B on rollers. On any bar (or fiber, if there is flexure) of the statically determined structure, let

S_0 = stress due to the actual loads;
t = " " " an outward load of unity at B;
r_1 = " " " a right-handed moment of unity at B;
r_2 = " " " " left-handed " " " " C.

Let the actual thrust (H_0), the moment at B (M_1), and the moment at C (M_2), be assumed to act as shown. Then the equations of Case VI apply here, and the solution is given by Equations (19), (20), and (21), all the quantities in those equations being easily found by statics.

The further discussion and illustration of these results are interesting and valuable, but need not be given here, as the purpose is merely to explain the new principle which, if not first stated, is believed to be first explained, in this paper.

It is not the intention here to do more than explain the principle and illustrate its application. The reader who thoroughly grasps it will find that it may be applied usefully in many ways.

Case VIII.—Secondary Stresses.—The principle herein explained may be applied to the determination of the secondary stresses in framed structures, in the following manner:

Let the structure be a frame with assumed frictionless connections, and with bars arranged so as to be statically determined as regards the inner forces. The secondary stresses arise from the fact that, if the joints are riveted, they are far from frictionless, so that unknown bending moments are introduced at the ends of the bars. There is such a bending moment at each end of each bar.

Let $n =$ number of bars,
$m =$ " " joints.

Then, the structure being statically determined with regard to the inner forces, the condition is fulfilled

$$n = 2m - 3$$

Now, as regards the secondary stresses, the only unknown quantities are the bending moments at the ends of the bars, so that

$$x = \text{number of unknown quantities} = 2n.$$

The conditions from which these unknown quantities may be found are, that with rigid connections the angles between the bars at any given joint are unchanged, or, in other words, that the rotation of every bar meeting at a joint is the same.

At all but four of the joints there will be, in general, four bars meeting, if the frame is a simple triangular frame, so that at each such joint there will be three equations. At two joints there will be three bars meeting, and two equations; and at two joints there will be two bars meeting, and one equation. In addition, there is the condi-

tion at each joint that the algebraic sum of the moments at the ends of the bars meeting there must be zero. Hence, the number of equations will be

$$3\ (m - 4) + 4 + 2 + m = 4m - 6 = 2n$$

so that there are just equations enough to determine the unknown quantities.

The foregoing holds thus far for any frame composed of simple triangles like a Warren truss, in Fig. 13, but not for such a truss with secondary bars; but, if the method of developing the truss with secondary bars is considered, the principle will be found to be true for such a truss also.

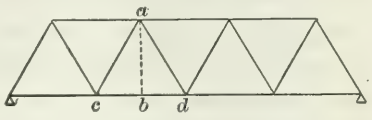


FIG. 13.

Consider the simple Warren truss, as shown in Fig. 13. If, now, it is desired to add a secondary bar, *ab*, by connecting an existing joint, *a*, with some point, *b*, in another bar, *cd*, one new joint is added (*b*), and two new bars (*ab* and *cb* or *bd*). For the original truss

$$n = 2m - 3.$$

If m_1 secondary bars are added, such as *ab*, there will be $2m_1$ new bars added, so that

$$\begin{aligned} n_0 = \text{number of bars} &= n + 2\ m_1 = 2\ m - 3 \\ &+ 2\ m_1 = 2\ (m + m_1) - 3 \\ m_0 = \text{number of joints} &= m + m_1 \end{aligned}$$

and, as before, $n_0 = 2\ m_0 - 3$, or the structure is still statically determined.

As regards secondary stresses, the number of conditions in the original truss was $4\ m - 6$. For each secondary bar added, there is a joint, *b*, added, at which there are three bars meeting, and two conditions, hence there will be $2\ m_1$ additional conditions if m_1 secondary bars are added. Also, at each of these joints, *b*, $\Sigma\ M = 0$, which gives m_1 more conditions; also, at each joint, *a*, there is one more bar and hence one more condition, which gives m_1 more conditions. Hence, the addition of m_1 secondary bars gives $4\ m_1$ additional conditions, making the total number of conditions $4\ m - 6 + 4\ m_1$. The number of bars will be $n + 2\ m_1$, and the number of unknown quantities will be

$$2\ (n + 2\ m_1) = 4\ m - 6 + 4\ m_1$$

or the same as the number of conditions.

All the stresses due to the moments at the ends of the bars may be easily computed by statics, and the resulting numerical equations may be solved, in any given case, though general formulas would be difficult to deduce, and are unnecessary.

The effect of eccentricity of the connections may be taken account of by the same method. The stress in any bar is assumed to act in the line joining the ideal joints of the truss. If this line is not through the center of gravity of the cross-section of the bar, it causes a moment at each end, which is a part of the total moment at that end.

In the same way, the effect of pin connections may be taken account of. The friction on the pin when the bar tends to rotate on it as the truss deflects, will cause a moment at the end of the bar, depending on the coefficient of friction and the diameter of the pin. The structure may be computed, first, as if the joints were rigid, and the moment at the end of the bar found. If this moment proves to be less than the frictional moment, the bar will then be actually rigidly fixed at the end, notwithstanding the pin connection. If the moment as found proves to be greater than the frictional moment, the latter is to be taken as the true moment, and the results throughout the truss modified accordingly.

DISCUSSION

Mr. Hudson.

C. W. Hudson,* M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest and has verified the derivations of the numbered equations. The general statement involved in Equation (3) he has not seen before, and the same may be said of Equation (4). In the short time in which the writer has studied the possibility of the application of Equation (4) to determine the angular rotation of any normal section of a bar, he is unable to see that it has any advantage, from the labor-saving standpoint, in shortening or abbreviating such calculations. Equations (1) and (2) and the further well-known equation, $\Delta \alpha = \int \frac{M\, d\, s}{E\, I}$, furnish the basis for the solution of all the problems which may be solved by Equation (4), and it is believed by the writer that the resulting solution of many problems would be the same, term for term, in either case. As the author states, it is often desired to find the angular rotation of the axis at a definite point of a bar subject to flexure. This, of course, is the same in direction and amount as the angular rotation of a section normal to the axis at the point. For about 16 years the writer has been using Equation (2) to determine the angular rotation of certain planes for which the author shows the application of Equation (5). If the auxiliary force of unity, imagined in the application of the general Equation (2), is considered as applied parallel to the axis of a straight bar at any cross-section of the bar and at a unit's distance from the axis, the deflection at the point of application of the unit force, divided by unity, is the angular rotation of the cross-section, in radians. As illustrations:

Equations (7) and (8) of Case I are seen to follow directly from Equation (2), with the aid of Fig. 14.

For the various equations derived for Case II: Equation (9) will be seen to follow directly from Equation (2), in connection with Fig. 15.

The slope at the load (α_l) is obtained from Equation (2), in connection with Fig. 16.

The point where the axis is horizontal may be found from Equation (2), in conjunction with Fig. 17.

The slope at the right end, given by Equation (10), due to the moment, M, at the left end causing compression in the top fibers of Fig. 5, may be found from Equation (2) and the data of Fig. 18.

The slope at the right end, given by Equation (11), due to a moment, M_2, at the right end causing compression in the top fibers of Fig. 6, may be found from Equation (2) in conjunction with Fig. 18.

The results obtained by the use of Equation (2) are identical with those obtained by the use of Equation (5) in each of the foregoing, and it would appear that Equation 5 is only a special case of Equation (2).

* New York City.

Mr. Hudson.

The author's derivation of the theorem of three moments under Case III follows directly from the relations established for the various subdivisions of Case II, and, therefore, may be said to be derived wholly by the use of Equation (2). There are numerous forms of the three-moment equation. In the latter part of 1916, the writer derived, and published,* the following well known three-moment equation as well as several other three-moment equations, by means of the angular rotation of the vertical plane section over any intermediate support of a continuous beam. The derivations referred to and the following are from Equation (2). Let Fig. 19 represent any two adjacent spans of a continuous beam resting on level supports. Let the nature of the supports be such that the beam is without restraint in a longitudinal direction.

Let Fig. 20 show the left span of Fig. 19, on which the forces and moments shown are those existing when this span is acting as a part of the whole beam. In order to find the angular motion of the end planes of this span of the beam, the auxiliary load of unity will be considered to be applied at a unit's distance from the axis.

For determining the angular rotation of the left end, the auxiliary force of unity, and the reactions produced by it, are shown in Fig. 21.

The clockwise motion of the left end is:

$$\alpha_{2l} = \int_0^{l_2} -M_2 \left(1 - \frac{x}{l_2}\right) \frac{d\,x}{E\,I_2} + \int_0^{l_2} -\frac{(M_3 - M_2)}{l_2}\, x \left(1 - \frac{x}{l_2}\right) \frac{d\,x}{E\,I_2}$$

$$+ \int_0^{l_2} P_2\,(1 - k_2)\, x \left(1 - \frac{x}{l_2}\right) \frac{d\,x}{E\,I_2}$$

$$+ \int_{k_2 l_2}^{l_2} - P_2\,(x - k_2\,l_2) \left(1 - \frac{x}{l_2}\right) \frac{d\,x}{E\,I_2}$$

$$= \frac{M_2}{E}\left[- \int_0^{l_2} \frac{d\,x}{I_2} + \int_0^{l_2} \frac{2\,x\,d\,x}{I_2\,l_2} - \int_0^{l_2} \frac{x^2\,d\,x}{I_2\,l_2^2} \right] + \frac{M_3}{E}\left[- \int_0^{l_2} \frac{x\,d\,x}{I_2\,l_2} \right.$$

$$\left. + \int_0^{l_2} \frac{x^2\,d\,x}{I_2\,l_2^2} \right] + \frac{P_2}{E}\left[+ \int_0^{k_2 l_2} \frac{x\,d\,x}{I_2} + k_2 \int_0^{k_2 l_2} \frac{x\,d\,x}{I_2} - \frac{1}{l_2} \int_0^{k_2 l_2} \frac{x^2\,d\,x}{I_2} \right.$$

$$\left. + k_2\,l_2 \int_{k_2 l_2}^{l_2} \frac{d\,x}{I_2} - 2\,k_2 \int_0^{l_2} \frac{x\,d\,x}{I_2} + \frac{k_2}{l_2} \int_0^{l_2} \frac{x^2\,d\,x}{I_2} \right] \ldots\ldots (D)$$

Fig. 22 shows the auxiliary force of unity, and its reactions, necessary to determine the clockwise motion of the right-end plane which is, in the same manner as before:

$$\alpha_{2r} = \frac{M_2}{E}\left[\int_0^{l_2} \frac{x\,d\,x}{I_2\,l_2} - \int_0^{l_2} \frac{x^2\,d\,x}{I_2\,l_2^2} \right] + \frac{M_3}{E} \int_0^{l_2} \frac{x^2\,d\,x}{I_2\,l_2^2} + \frac{P_2}{E}$$

$$\left[- \frac{1}{l_2} \int_0^{k_2 l_2} \frac{x^2\,d\,x}{I_2} + \frac{k_2}{l_2} \int_0^{l_2} \frac{x^2\,d\,x}{I_2} - k_2 \int_{k_2 l_2}^{l_2} \frac{x\,d\,x}{I_2} \right] \ldots\ldots (E)$$

* In the *Polytechnic Engineer* for 1917.

Mr. Hudson.

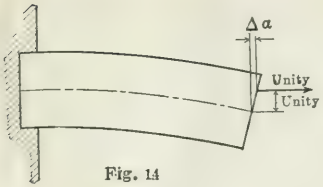


Fig. 14

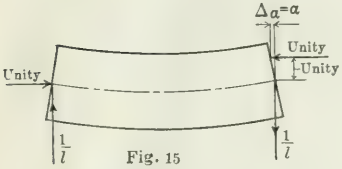


Fig. 15

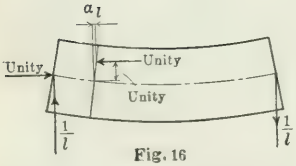


Fig. 16

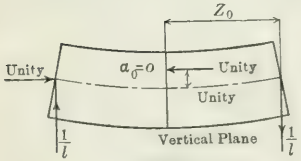


Fig. 17

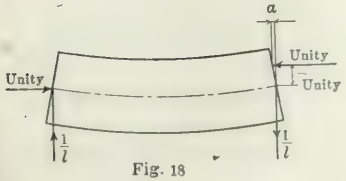


Fig. 18

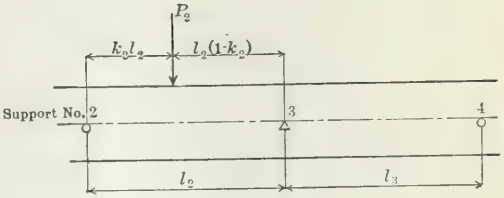


Fig. 19

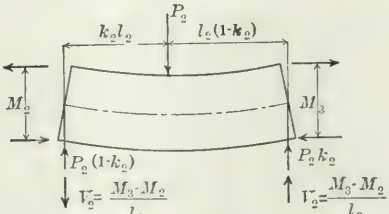


Fig. 20

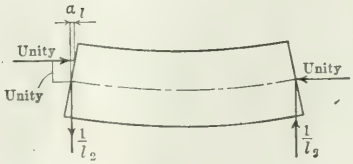


Fig. 21

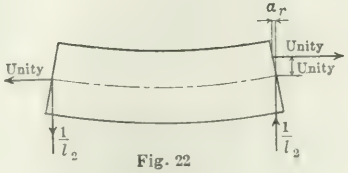


Fig. 22

Mr. Hudson.

For the right-hand span of Fig. 19, the angular changes of the end planes under the same assumptions as for the left span are:

$$\alpha_{3l} = \frac{M_3}{E}\left[-\int_0^{l_3}\frac{d\,x}{I_3} + \int_0^{l_3}\frac{2\,x\,d\,x}{I_3\,l_3} - \int_0^{l_3}\frac{x^2\,d\,x}{I_3\,l_3^2}\right] + \frac{M_4}{E}\left[-\int_0^{l_3}\frac{x\,d\,x}{I_3\,l_3} + \int_0^{l_3}\frac{x^2\,d\,x}{I_3\,l_3^2}\right] + \frac{P_3}{E}\left[+\int_0^{k_3 l_3}\frac{x\,d\,x}{I_3} + k_3\int_0^{k_3 l_3}\frac{x\,d\,x}{I_3} - \frac{1}{l_3}\int_0^{k_3 l_3}\frac{x^2\,d\,x}{I_3} + k_3 l_3\int_0^{l_3}\frac{d\,x}{I_3} - 2\,k_3\int_0^{l_3}\frac{x\,d\,x}{I_3} + \frac{k_3}{l_3}\int_0^{l_3}\frac{x^2\,d\,x}{I_3}\right] \ldots (F)$$

$$\text{and } \alpha_{3r} = \frac{M_3}{E}\left[\int_0^{l_3}\frac{x\,d\,x}{I_3\,l_3} - \int_0^{l_3}\frac{x^2\,d\,x}{I_3\,l_3^2}\right] + \frac{M_4}{E}\int_0^{l_3}\frac{x\,d\,x}{I_3\,l_3^2} + \frac{P_3}{E}\left[-\frac{1}{l_3}\int_0^{k_3 l_3}\frac{x^2\,d\,x}{I_3} + \frac{k_3}{l_3}\int_0^{l_3}\frac{x^2\,d\,x}{I_3} - \frac{k_3}{l_3}\int_{k_3 l_3}^{l_3}\frac{x\,d\,x}{I_3}\right] \ldots\ldots (G)$$

When the beam is acting as a whole, α_{2r} equals α_{3l}, as they are angular rotations of the same plane; therefore, we may write the right half of Equation (E) equal to the right half of Equation (F). Before writing the equation it should be noted that M_2, M_3, and M_4, have been used, as shown in Fig. 20, as negative moments, hence their values as determined from any combination of the foregoing equations would be positive. Therefore, it will be best to change the signs of the moments over the supports to make them agree with the general convention. By doing this, there results the following equation of three moments:

$$M_2\left[-\int_0^{l_2}\frac{x\,d\,x}{I_2\,l_2} + \int_0^{l_2}\frac{x^2\,d\,x}{I_2\,l_2}\right] - M_3\int_0^{l_2}\frac{x^2\,d\,x}{I_2\,l_2^2} + M_3\left[-\int_0^{l_3}\frac{d\,x}{I_3} + \int_0^{l_3}\frac{2\,x\,d\,x}{I_3\,l_3} - \int_0^{l_3}\frac{x^2\,d\,x}{I_3\,l_3^2}\right] + M_4\left[-\int_0^{l_3}\frac{x\,d\,x}{I_3\,l_3} + \int_0^{l_3}\frac{x^2\,d\,x}{I_3\,l_3^2}\right] = -P_2\left[-\frac{1}{l_2}\int_0^{k_2 l_2}\frac{x^2\,d\,x}{I_2} + \frac{k_2}{l_2}\int_0^{l_2}\frac{x^2\,d\,x}{I_2} - k_2\int_{k_2 l_2}^{l_2}\frac{x\,d\,x}{I_2}\right] + P_3\left[\int_0^{k_3 l_3}\frac{x\,d\,x}{I_3} + k_3\int_0^{k_3 l_3}\frac{x^2\,d\,x}{I_3} + k_3 l_3\int_{k_3 l_3}^{l_3}\frac{d\,x}{I_3} - 2\,k_3\int_0^{l_3}\frac{x\,d\,x}{I_3} + \frac{k_3}{l_3}\int_0^{l_3}\frac{x^2\,d\,x}{I_3}\right] \ldots\ldots\ldots\ldots\ldots\ldots (H)$$

Equation (H) may be used to design or investigate continuous girders having a varying moment of inertia. A somewhat simpler form of Equation (H) may be derived by measuring x and k from the left end of the left span and the right end of the right span of the two spans adjacent to any support. The differences in elevation of the supports may readily be included in the foregoing, provided they are small, as it is entirely a matter of geometry.

Case IV is in reality only a repetition of Case II. Mr. Hudson.

Case V is more readily solved by Equation (1) than by Equation (4), as the author shows.

With respect to the remaining cases, suggested for illustration by the author, the writer believes, that Equations (1) and (2) will give simpler solutions than Equation (4), with the possible exception of Case VIII. For the latter, the value of the method can only be determined by considerable investigation, and this the writer has not made.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—The formula given in Equation (1) is familiarly known as the "Sigma $p\ u\ l$ over E" formula, though that equation is not in the same shape as this easily remembered form. In the familiar form it is Mr. Godfrey.

$$\Delta = \Sigma \frac{p\ u\ l}{E}$$

where Δ is the deflection of any point in a frame in a given direction;

p is the stress in any member of the frame due to a load of 1 lb. in that same direction and applied at the same point;

u is the unit stress in the member under its stress;

l is the length of the member;

and E is the modulus of elasticity.

This is a useful equation and the form just given is a simple one to remember and apply. The writer has used it in dealing with drawbridges and hingeless arches, and it is needed for such indeterminate cases.

The author's new formula may also be found useful for indeterminate cases. It might be suggested that it be simplified to the form

$$\alpha = \Sigma \frac{r\ u\ l}{E}$$

where α is the angular rotation in a frame in any point in a given direction;

r is the stress in any member of the frame due to a couple of unity, applied at the point in question, that will tend to give rotation in that same direction;

and the other values are as before.

This could be called the "Sigma $r\ u\ l$ over E" formula.

The writer believes there is an error in Equation (7) in which the inclination at the end of a cantilever beam is given as $\frac{P\ l^2}{E\ I}$, and that it should be $\frac{P\ l^2}{2\ E\ I}$.

* Pittsburgh, Pa.

Mr. Godfrey.

Although the writer does not question the correctness of the formulas nor the possibilities of their usefulness in a limited way, he thinks that their application to a king-post truss and framed bents, and to secondary stresses from such causes as rotation of members on the pins due to deflection or to rigidity of riveted connections, although interesting as studies in what might be called pure applied mathematics, have no place in practical engineering.

The reason for this is twofold, and both phases deserve consideration and emphasis. Stating the reasons briefly: (1) Perhaps not one engineer in fifty doing practical designing could follow this paper or apply it in a practical design; (2) There is no reason on earth why it should be applied in a practical design.

Taking these up separately: Is there any reason for not simplifying mathematics so that the average designer can understand and apply it? The writer does not hesitate to state that, if it is a matter of the designing of ordinary construction, the design should be simplified down to the last degree. There are some structures, such as suspension bridges and fixed-ended arches, that come up only once in a long time, and these can be referred to a mathematical expert. Such matters, however, as mill building and truss design are the daily concern of men who have only a limited knowledge of mathematics. They cannot and will not use the higher methods of designing, even if these are required in safe design; and the sad fact is that they will not, and do not, use ordinary common sense at all times, with the result that a large number of poor designs are turned out.

The writer, during the past 25 years, has been in a position in which few men in the Profession are placed, and is able to make the statement in the foregoing paragraph with emphasis. Called on repeatedly, as he is, to check the designs of others, both recent and ancient, he is thoroughly convinced that what designers need is not more complicated mathematical processes, not more mathematics, but, in fact, less mathematics and more common sense in instruction in designing.

The design of a pair of columns supporting a roof truss, with knee-braces, can be made an exceedingly complicated problem. It can be made so intricate that the average designer could not touch it. Only an expert, and that expert devoting many hours to the problem, could run out all the possibilities of stress of this simple piece of construction. The building would not stand the cost of engineering of this class. Here is the economic situation. What is the outcome? The complex treatment is thrown to the winds, and the truss and columns are designed by rule-of-thumb, and bad rule at that. The columns are designed for direct load only, with no regard to bending moments or wind loads. Frequently, no knee-braces whatever are used, and a flimsy and wabbly building results.

Mr. Godfrey.

On the other hand, the solution of this mill building problem can be rendered as simple as an ordinary truss design by ignoring trivial theoretical matters, by dividing equally between the columns the horizontal force of the wind, and computing the force in the knee-braces and the truss from these applied loads; and a building designed in this manner is both economical and perfectly safe. The average designer can grasp this method quite readily; but our books and engineering literature delight in investing this problem with the greatest complexity, with the result that the majority of designers do not even attempt to approximate its solution, and bad structures grow out of this condition of affairs.

If a fraction of the space taken up with fancy solutions of the mill building problem was devoted to some common-sense simple methods of getting the stresses approximately, and some homely advice was given on how to detail parts, the Profession would be much better off. For example, the writer wished to find (for the head of an engineering school) the details of a mill building where the chords intersect over the support, as an example of good practice in detailing. A thorough search, and inquiry among bridge firms, failed to secure a single case.

As to the question of the need of these higher theories, the mere fact that they are ignored habitually in the design of mill buildings does not argue one way or the other. It is notorious that mill buildings are poorly designed. Much other structural designing is also poor, not because finer theories are neglected, but because common sense is not exercised. The writer would never dream of applying the theory of secondary stresses to a building or bridge, but he has found errors by the hundred, in both design and detail, that would not be possible if designers were instructed in the simple rudiments of proper design that require a knowledge of nothing but the simplest mathematical processes and ordinary mechanics. It is not important for a designer to find that a detail needs 21.1 rivets or 21.3 rivets, but, when a design is turned out and has 11 rivets where about 21 are required, something should be done that will put that designer on the right track.

The members of a cantilever bracket may intersect in the exact center of a truss, just as theory may dictate, and still the bracket may be dependent on the tension in the heads of three or four rivets. This is higher theories with a vengeance!

If steel was perfectly elastic and perfectly brittle, like glass, higher theories would be appropriate and no doubt necessary. Then secondary stresses would loom large in all design. The great Firth of Forth Bridge would then be one of the most dangerous bridges in the world, because of the great secondary stresses due to deflection; but, because steel is tough and not perfectly elastic, that bridge is one of the safest in existence. The tubular members and the rigidly riveted details

Mr. Godfrey.

are the very things that make it so safe, and they are the very things that, theoretically, add tremendously to the stresses. Will engineers ever get this idea?

It is a strange coincidence that the Royal Commission that investigated the Quebec Bridge failure attributed its weakness to the fact that the connections of the compression members were not riveted up. The condition for creating secondary stresses was absent. The same Royal Commission ignored the big traveler, which, being without bracing, upset and pulled the bridge over with it. It does not take even a knowledge of algebra to understand this glaring error.

The Orpheum Theater, in New York City, collapsed, and, in all the reports of its failure, the real cause was ignored. This was a monstrous curved girder, with a span of 74 ft. and a versed sine of 12 ft., connected at the ends to the corners of slender columns.* Among the theories advanced to account for the failure was one, from across the ocean, that attributed it to the twisting in a supporting girder due to deflection of the cross-girders. This is the very limit. It is the theory of secondary stresses carried to its logical limit of absurdity. It is hard to think of anything more misleading and harmful than this kind of theoretical camouflage, not because theoretical speculation in itself is harmful, but because it obscures the facts and takes up space that might be devoted to constructive knowledge of a simple character.

The foregoing instances are given, not because the writer would deprecate the value of theory in engineering design, but would elevate it to a position of usefulness, pure and simple. He would remove it from the condition where it is virtually merely an academic plaything, applied where it has no meaning whatever, and on premises of properties of materials that do not exist in fact. He would make theory agree with facts as they are, and not with conceptions of what might exist if steel and concrete had other properties than they are known to possess; and, lastly, he would simplify theory, so that it is understandable on the part of the men who use it.

Mr. Church.

IRVING P. CHURCH,† ASSOC. AM. SOC. C. E. (by letter).—The writer has been much interested in examining this paper with a view to ascertaining to what extent it supplements the principles and methods proved and exemplified by Castigliano in his classic book‡ on elastic framed systems, or mechanics of the "work of elastic deformation", or of "internal work", as it might also be called.

A very brief résumé of the main theorems proved by Castigliano, therefore, will be undertaken; the elastic framed structure in question being a jointed assemblage of bars and beams (straight or curved), fitted

* *Engineering and Contracting*, February 18th, 1914.

† Ithaca, N. Y.

‡ This book, by A. Castigliano, is entitled "Théorie de l'Equilibre des Systèmes Elastiques", and was published at Turin in 1879.

Mr. Church.

together originally without strain, with the axes of all members lying in a common vertical plane (plane of loading), and with all cross-sections of members symmetrical about that plane.

The notation will be that of the author, with a few supplementary symbols.

Let U denote the total work of elastic deformation (*i. e.*, "internal work") of the frame when loaded. Then, omitting the work of shear, as of very slight consequence, we have

$$U = \sum \int \frac{T_s^2 \, d\,l}{2\,E\,A} + \sum \int \frac{M_s^2 \, d\,l}{2\,E\,I} \quad \ldots\ldots\ldots\ldots (24)$$

Also, the deflection (displacement) of any point of any member, or of a joint, at which point or joint a load or force, P, is applied, this deflection being that taking place in the direction of the line of action of such force or load, will be given by the relation

$$\Delta = \frac{d\,U}{d\,P} = \sum \int \frac{T_s}{E\,A} \left(\frac{d\,T_s}{d\,P} \right) d\,l + \sum \int \frac{M_s}{E\,I} \left(\frac{d\,M_s}{d\,P} \right) d\,l \ldots (25)^*$$

The first term in the right-hand member of Equation (25) is of a general form, so as to provide, not alone for straight bars in which there is direct stress only, but also for bars or beams in which, or in portions of which, T_s or A may not be constant along the length.

For a frame of ordinary straight bars under direct stress of tension or compression, as in a pin-connected bridge truss, Equation (25) reduces to

$$\Delta = \sum \frac{T_s\,l}{E\,A} \left(\frac{d\,T_s}{d\,P} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Similarly, the change of slope, or angular rotation, due to the loading, of the straight line joining any two points of the frame (not joints necessarily) where there are applied two equal parallel forces forming a couple, of moment M, the two forces being at right angles to that line, which is, therefore, the "arm" of the couple, is determined by the relation

$$\alpha = \frac{d\,U}{d\,M} = \sum \int \frac{T_s}{E\,A} \left(\frac{d\,T_s}{d\,M} \right) d\,l + \sum \int \frac{M_s}{E\,I} \left(\frac{d\,M_s}{d\,M} \right) d\,l \ldots (27)$$

(See foot-note following Equation (25)).

Castigliano goes on to treat the case of a frame containing one more bar than is necessary for the frame to hold its shape; this "superfluous", or "redundant", bar being pin-connected at its extremities and of such length that it is too short, or too long, by a small amount,

* A very important restriction on the use of Equation (25), and the other fundamental relations to follow, is that the supporting forces (or reactions) acting on the frame must be "neutral"; that is, the point of application of each such force must not move during the gradual application of the loading, unless along a line at right angles to the line of action of such force.

Mr. Church.

λ_0, to fit into its place after the "necessary" members have been assembled without loading; so that, to fit it into place, special constraint must be exerted. After this "redundant" bar is in place and the frame is loaded, the special constraint having been removed, the direct stress, T' produced in the bar (which carries no load along its length) is found by the relation

$$\frac{d\,U}{d\,T'} = \lambda_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Hence, it follows, as a particular case, that, when the "redundant bar" has just the proper length to fit into place originally without strain, the direct stress, T, induced in it when the loading is in position, is determined by the equation

$$\frac{d\,U}{d\,T} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

This has been called the principle of "least work"; as it may easily be proved, by the consideration of a curve (parabola with axis vertical), plotted with U for ordinate and T (as variable) for abscissa, that the special value of T found from Equation (29) is the abscissa of the lowest point of the curve, where the tangent line is horizontal, and, consequently, the corresponding value of U is less than for any other (tentative) value of T.

For finding the deflection, in a given direction, of a point in the frame where there is no load, or at least no load acting in the desired direction, Castigliano uses the device of inserting an imaginary load, P_0, at the point in question and in the desired direction; and later reduces it to zero (as, of course, must finally be done, for correct results). This auxiliary load or force, P_0, which may be called a "dummy" load, and is treated as a variable, will enter more or less into the expressions for T_s and M_s, the differentiation of which expressions with respect to P_0 will give the derivatives, $\left(\frac{d\,T_s}{d\,P_0}\right)$ and $\left(\frac{d\,M_s}{d\,P_0}\right)$. Now, these derivatives will not contain P_0, because, for ordinary frames, only the first power of P_0 can occur in any term of the expressions for T_s and M_s, and hence are not affected when the value of P_0 is made zero. Equation (25) is then used for finding the deflection, with $P_0 = 0$; that is, by substituting in it the foregoing values of the derivatives together with the values of T_s and M_s, as determined from the actual loading alone, P_0 being omitted. Hence, in this case of the "dummy" load, P_0, we have

$$\Delta = \frac{d\,U}{d\,P_0} = \sum \int \frac{T_s}{E\,A}\left(\frac{d\,T_s}{d\,P_0}\right) d\,l + \sum \int \frac{M_s}{E\,I}\left(\frac{d\,M_s}{d\,P_0}\right) d\,l \ldots (30)$$

Mr. Church.

Now $\frac{d\,T_s}{d\,P_0}$ $\left(\text{or } \frac{d\,M_s}{d\,P_0}\right)$ is the rate at which T_s (or M_s) varies with respect to P_0, at a rate of so many pounds (or foot-pounds) per pound of P_0; and furthermore, as T_s (or M_s) is a straight-line function of P_0 (for ordinary "engineering" frames), the value of the rate, $\frac{d\,T_s}{d\,P_0}$, $\left(\text{or } \frac{d\,M_s}{d\,P_0}\right)$, which represents "the slope" of the straight line, is numerically the same as the hypothetical value that T_s (or M_s) would have if P_0 were the only load and had a value of unity.* These hypothetical values may be called T_1 and M_1, respectively. Hence, a ready way of expressing the values of the two derivatives, $\frac{d\,T_s}{d\,P_0}$ and $\frac{d\,M_s}{d\,P_0}$, is to write out what T_s and M_s would be if a unit load were applied at the point in question and in the desired direction, with no other load on the frame. With this change of notation, therefore, Equation (30) becomes

$$\Delta = \sum \int \frac{T_s\,T_1}{E\,A} \cdot d\,l + \sum \int \frac{M_s\,M_1}{E\,I} \cdot d\,l \ldots\ldots\ldots (31)$$

(Castigliano does not use the device of a unit load for finding deflections. It was discovered and adopted independently by the author of this paper in America and by Fraenkel in Europe.)

Castigliano not only uses the device of a "dummy" load for finding deflections, but also that of an imaginary, or "dummy", couple, of moment M_0 (say), applied to the free extremity of a beam in order to find the change of slope, due to the loading, of the tangent to the elastic curve of the beam at that extremity, there being no actual couple at that point. M_0 is finally reduced to zero. As in the previous case of the "dummy" load, T_s and M_s are expressed in terms of M_0 and the other loading, so that the derivatives, $\frac{d\,T_s}{d\,M_0}$ and $\frac{d\,M_s}{d\,M_0}$, may be determined. According to Equation (27) the formula for the change of slope (or rotation) at that point, with the couple in place, is

$$\alpha = \frac{d\,U}{d\,M_0} = \sum \int \frac{T_s}{E\,A}\left(\frac{d\,T_s}{d\,M_0}\right) d\,l + \sum \int \frac{M_s}{E\,I}\left(\frac{d\,M_s}{d\,M_0}\right) d\,l \ldots (32)$$

and when, in this relation, M_0 is made equal to zero, the derivatives retain their values unchanged, while T_s and M_s assume the values proper to them under the actual loading alone, the couple having been removed. As before, as T_s and M_s are almost invariably straight-line functions of M_0, the numerical values of these derivatives are equal, respectively, to the values that T_s and M_s would have if the "dummy" couple had a moment equal to unity and constituted the sole loading on

* Or, to put it another way, the finite change or increment of T_s (or M_s) due to the introduction of P_0 among the loads is proportional to P_0.

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the frame or beam. Hence, these derivatives become the T_r and M_r, respectively, of the paper, and the result will now read

$$\alpha = \sum \int \frac{T_s T_r}{E A} . d l + \sum \int \frac{M_s M_r}{E I} . d l \ldots\ldots (33)$$

which is the author's Equation (6). This equation is given and proved by Müller-Breslau* in connection with straight and curved beams. He makes very little use of it, however.

In his treatment of single beams, Castigliano rarely finds occasion to use a "dummy" couple, to determine the rotation of a tangent line of the elastic curve; because, when any cross-section forms the extremity of a portion of the beam, considered as a complete beam or frame in itself, apart from the remainder (as, for instance, in dealing with the continuous girder), the flexural couple of the cross-section becomes a part of the external loading for that portion, and is used by Castigliano in expressing the change of slope at that section; in other words, no "dummy" couple is needed in such a case.

By way of a comparative showing of these three methods for finding the change of slope, or rotation, of a tangent line at a definite point of a loaded beam, take the simple case of a straight elastic prismatic beam, fixed horizontally at the left-hand end and loaded at the other (free) extremity with a vertical load, P, and determine the rotation, α, of the tangent at the midway point of the length, from its original horizontal position.

First Method.—Dummy Couple.—Fig. 23.—Conceive a rigid straight bar, with its axis following the tangent in question, to be fastened to the beam at the point, B, by two bolts very near together, and a couple, of moment M_0, to be applied to it, this couple being right-handed, so as to tend to produce a value of α of the same character as that due to the load, P. This bar forms an additional member of the elastic system, and two consequences follow from its being rigid: (1) its axis will remain straight during the gradual loading, always coinciding with the tangent at B, so that the rotation of this "arm" of the couple is the same as that of the tangent; (2) the "internal work" of the bar is zero, so that no terms are needed in Equation (32) to cover the deformation of the bar (that is, its E is infinite).

In Equation (32), therefore, we have: For the elements of length, dx, along the portion, BC:

$$T_s = 0; \text{ whence } \frac{d T_s}{d M_0} = 0;$$

$$M_s = -Px; \text{ whence } \frac{d M_s}{d M_0} = 0;$$

* It is Equation (54a) on p. 133 (see also pp. 69 and 205) of his work entitled: "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," Leipsic, 1893, this equation containing also terms for dealing with a possible change of temperature and slight movement of supports.

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and along the portion, $A\ B$:

$$T_s = 0;\ \text{whence}\ \frac{d\ T_s}{d\ M_0} = 0;$$

$$M_s = -P\ (a + x) - M_0;\ \text{whence}\ \frac{d\ M_s}{d\ M_0} = -1.$$

Hence, making $M_0 = 0$ and substituting in Equation (32),

$$\alpha = 0 + 0 + 0 + \frac{1}{E\ I}\int_0^a [-P\ (a + x)]\ (-1)\ d\ x \ldots\ldots (33)$$

that is,

$$\alpha = \frac{P}{E\ I}\left(a^2 + \frac{a^2}{2}\right) = \frac{3}{2} \cdot \frac{P\ a^2}{E\ I} \ldots\ldots\ldots\ldots (34)$$

or, as $a = \frac{1}{2}\ l$,

$$\alpha = \frac{3}{8} \cdot \frac{P\ l^2}{E\ I};$$

as may easily be found by more elementary methods.

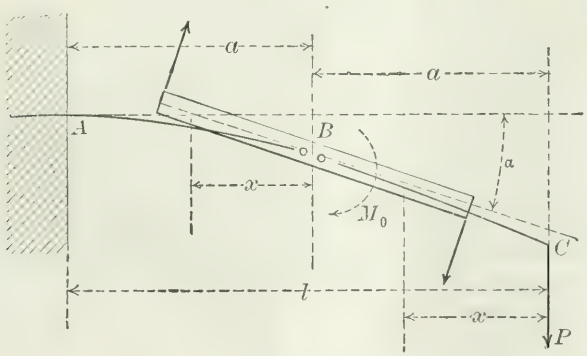

FIG. 23.

Second Method.—Utilizing an Internal Couple.—Fig. 24.—The internal couple, or flexural couple, at section B has a moment we may call M_B, and if the portion, AB, of the beam is considered as a complete elastic system in itself ("free body"), the couple becomes an external couple, as this couple and the vertical shear, J_B, of the section constitute a loading representing the action of the part, BC, conceived as removed, upon the extremity, B, of the portion remaining, AB, now considered in Fig. 24. The flexural couple and shear at A play the part of supporting forces, but their points of application do not move, and this fact justifies the use of Castigliano's theorems.

(Equation (27) could not be applied to the portion, BC, considered by itself, for finding α, because, of the four forces acting on that

Mr. Church. portion, no three, viewed as supporting forces, have motionless points of application. See foot-note following Equation (25)).

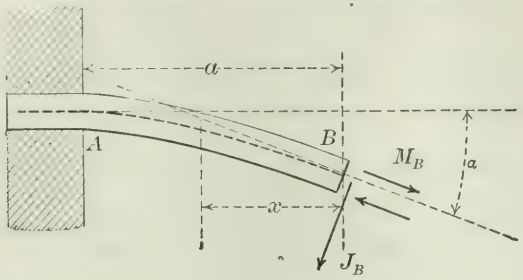

FIG. 24.

For the dx's along AB, we have

$$T_s = 0; \text{ whence } \frac{d\,T_s}{d\,M_B} = 0;$$

$$M_s = -J_B\,x - M_B; \text{ whence } \frac{d\,M_s}{d\,M_B} = -1$$

and hence Equation (27) gives

$$\alpha = 0 + \frac{1}{E\,I}\int_0^a [-J_B\,x - M_B]\,(-1)\,d\,x \ldots\ldots\ldots (35)$$

but $M_B = Pa$, and $J_B = P$, and, therefore, $\alpha = \frac{3}{2}\cdot\frac{P\,a^2}{E\,I}$, as before.

Third Method.—Using an Imaginary Couple of Moment Equal to Unity.—Fig. 25.—Conceive a rigid cross-bar bolted to the beam at section B, and consider a couple applied to it of moment equal to -1 (minus, in order to have it in the same "sense" as the rotation, α, at B),

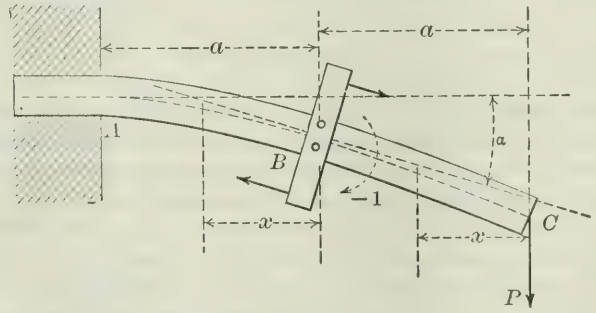

FIG. 25.

this being for the sole purpose of determining T_r and M_r for the successive infinitesimal lengths, dx, along the whole beam, for use in Equation (33).

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For any dx along BC

$$T_s = 0; \qquad T_r = 0;$$
$$M_s = -Px; \qquad M_r = 0;$$

and for any dx along AB

$$T_s = 0; \qquad T_r = 0;$$
$$M_s = -P\ (a + x); \quad M_r = -1;$$

and hence substitution in the author's Equation (6) or Equation (33) of this discussion, gives

$$\alpha = 0 + 0 + 0 + \frac{1}{EI}\int_0^a [-P\,(a + x)]\,(-1)\,dx \ldots\ldots (36)$$
$$\text{or } \alpha = \frac{3}{2} \cdot \frac{P\,a^2}{EI}$$

In conclusion, it would appear that, in finding the rotation of a tangent line, there is in many cases a decided advantage in using the device of a couple with a moment of unity, so far as the shortening of the routine work is concerned; and the author has rendered an important service in the publication of this paper, with its very direct and convincing proof and numerous applications.

Mr. Parcel.

JOHN I. PARCEL,* ESQ. (by letter).—The author has presented a clear and complete exposition of the rotational work equation, and some very interesting applications, notably the beautifully direct and simple derivation of the three-moment theorem in its general form.

The paper adds another to the many illustrations of the fact that widespread knowledge and full appreciation of new principles in the theory of structures frequently lags considerably behind their discovery. This seems to have been true regarding the "general work equations,"† the principle of "moment areas"‡ (its advocates assert that its scope and importance is still but half appreciated by the Profession generally), and the so-called "slope-deflection" method§, which has recently yielded elegant solutions of certain problems in rigid frames.

In so far as Professor Swain's new principle consists in expressing the angular movement at a given point in the axis of a bar by the method of work, in a manner analogous to that for linear movement,

* Associate Professor, Structural Eng., Univ. of Minnesota, Minneapolis, Minn.

† First stated in their general form by O. Mohr (1874), who also seems to have been the first to realize their vast importance in the analysis of structures. The essential features of Mohr's method, however, were anticipated (according to Müller-Breslau) by Maxwell (1864), and the fundamental principle was enunciated by Clapeyron in 1833.

‡ Generally ascribed to Mohr (1868), but developed independently in a somewhat different form by the late Charles E. Greene, M. Am. Soc. C. E. (1874). Its wide use in America is of comparatively recent date.

§ Applied by Mohr to the secondary stress problem as early as 1891 (*Zivilingenieur*, 1892, p. 577), but attracting little attention for many years. Presented as a general method of attack for a variety of problems in rigid frames by G. A. Maney ("Studies in Engineering, No. 1," University of Minnesota, 1915).

Mr. Parcel. it is not new. English and American treatises* on the theory of structures, as a rule, give a very restricted and inadequate presentation of the "method of work." German (and other continental European) treatises, however, lay considerable stress on the generality of the work equations, noting that Δ (to follow the notation of the author's Equations (1) and (2)) is linear or angular displacement, according as the "unit loading" which produces t and Mt is a unit force or a unit couple. Some of these works develop equations similar to the author's Equation (6), and give elementary examples of the calculation of angular change;† but the principle seems to be little understood (or at least little appreciated) generally, and Professor Swain has rendered no small service to the Profession by his excellent presentation.

Mr. Mensch. L. J. MENSCH,‡ M. AM. SOC. C. E. (by letter).—The principle of virtual displacements is one of the most general laws of equilibrium, and in it are contained the theory of least work, Maxwell's and Betti's reciprocal theorems, and Professor Swain's principle. These various rules are either identical or differ only in algebraic expressions of force, moment, and displacement.

The principle of virtual displacements says: The work performed by the exterior forces and reactions when acting on a structure in a state of equilibrium equals the work done by the interior forces in deforming the structure, which principle can be written in the form

$$\Sigma P \cdot \Delta = \Sigma T_s \cdot \Delta dl + \Sigma M_s \cdot \Delta d\alpha \ldots\ldots\ldots\ldots (1)$$

where P denotes the exterior forces and reactions;

Δ denotes their corresponding displacements at the point of application in the direction of the forces;

Δdl denotes the change of length of the length, dl, due to the normal forces, T_s, produced in the structure by the forces, P;

and $\Delta d\alpha$ denotes the change of angle of an element, dl, due to the moments, M_s, produced in the structure by the forces, P.

All expressions are in the familiar shape: Work equals force multiplied by displacement in the direction of the force; and the correctness of Equation (1) appears to be very plausible.

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* See, however, D. A. Molitor, M. Am. Soc. C. E., "Kinetic Theory of Engineering Structures", pp. 18 and 28–29. The exposition is much less complete than in the references cited below.

† See, especially, F. Hartmann, "Die Statisch Unbestimmten Systeme des Eisen und Eisen-betonbaues" (1913), pp. 9–11 and 24–33; also G. C. Mehrtens, "Statik und Festigkeitslehre" (1910), pp. 203–204 and 215 *et seq.*; H. F. B. Mueller-Breslau, "Die Graphische Statik der Baukonstruktionen", Band II, Erste Abtheilung (1903), pp. 14–17; and "Die Neueren Methoden der Festigkeitslehre" (1904), pp. 59 and 84–85. Mueller-Breslau seems to claim for himself the honor of the discovery of Professor Swain's new principle (see his "Graphische Statik", above cited, pp. 52–57). He states that in the 1886 edition of his "Festigkeitslehre", he first showed that the work equations (which he calls Maxwell's equations) apply to "rotation" as well as "translation."

‡ Chicago, Ill.

Mr. Mensch.

Clapeyron has shown that the actual work performed by gradually applied forces must be half of that above given, and the validity of Equation (1) is not changed by dividing each side by 2. Clapeyron has also shown that the validity of Equation (1) is not changed, when we substitute

First: for the forces and reactions, P, another set of forces, P', and their corresponding reactions, as long as the second set of forces and reactions hold the structure in a state of equilibrium; and

Second: The new forces, T', and the new moments, M', produced by the new set of forces and reactions, P', the displacements and deformations of the first set of forces being retained in Equation (1).

The new equation may be written

$$\Sigma P' \cdot \Delta = \Sigma T' \cdot \Delta dl + \Sigma M' \cdot \Delta d\alpha \ldots\ldots\ldots\ldots (2)$$

The new forces, P', and the new stresses, T' and M', have evidently nothing to do with, and are entirely independent of, Δ, Δdl, and $\Delta d\alpha$, of the first set of forces; their only co-relation is that each set belongs to a state of equilibrium; Δ, Δdl, and $\Delta d\alpha$, therefore, are called possible or virtual displacements. This is easily proved in the case of the simple structure shown in Fig. 26, consisting of three bars, supported at A and B and affected by a load, P, at C. The three bars undergo certain elongations and shortenings, producing a definite displacement at C. It is clear that any other load will produce stresses and displacements which are in direct proportion to the values of the two loads; and a load of $\frac{P}{2}$, for example, will produce half the stresses and half the changes of length, and also only half the displacement at C. For this case Equation (1) may be written, also, in the form

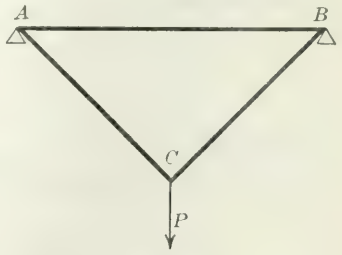

FIG. 26.

$$\Sigma \frac{P}{4} \Delta = \Sigma \frac{T}{4} \cdot \Delta dl.$$

$\frac{P}{4}$ and $\frac{T}{4}$ may be considered as new forces and stresses which are in a state of equilibrium, and Δ and Δdl are the old displacements. It was soon found that P, in Equation (1), and P', in Equation (2), are not necessarily forces, but may be moments, the latter being nothing else

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than a couple of equal forces acting at a distance, o, for example. We can say

$$M = P \cdot o;\ \Delta = o \cdot \alpha;\ M = P \cdot \frac{\Delta}{\alpha};\ M \cdot \alpha = P \cdot \Delta \ldots\ldots (3)$$

and Equations (1) and (2) become

$$\Sigma M \cdot \alpha = \Sigma T_s \cdot \Delta\, d\, l + \Sigma M_s \cdot \Delta\, d\, \alpha \ldots\ldots (4)$$

$$\Sigma M' \cdot \alpha = \Sigma T' \cdot \Delta\, d\, l + \Sigma M' \cdot \Delta\, d\, \alpha \ldots\ldots (5)$$

which equations express Professor Swain's idea in another form. Maxwell has deduced from Equation (2) a reciprocal theorem, relating to the effect produced in a structure by two different forces:

"A force, 1, in a point, A, of a structure in a state of equilibrium, will produce in any other point, B, of the same structure, a displacement, d_{ba}, which is equal to the displacement, d_{ab}, in the point, A, in the direction of the first force, 1, produced by another force, 1, acting in B in the direction, d_{ba}."

A more general theorem is due to Betti. Müller-Breslau claims that he has first shown that Maxwell's principle can also be applied to moments and to changes of angles, namely: "A moment, 1, acting in any point, A, produces a rotation in any point, B, which is equal to the rotation produced in A by a moment, 1, acting in B", which is, again, Professor Swain's principle expressed in another way.

Equations (2) and (5) can be transformed into a more useful shape by substituting

$$\Delta\, d\, l = \frac{T_s}{E\, A} \cdot d\, l \ldots\ldots (6)$$

$$\Delta\, d\, \alpha = \frac{M}{E\, I} \cdot d\, l \ldots\ldots (7)$$

thus obtaining

$$\Sigma P' \cdot \Delta = \Sigma \frac{T_s \cdot T' \cdot d\, l}{E\, A} + \Sigma \frac{M_s \cdot M' \cdot d\, l}{E\, I} \ldots\ldots (8)$$

$$\Sigma M' \cdot \alpha = \Sigma \frac{T_s \cdot T' \cdot d\, l}{E\, A} + \Sigma \frac{M_s \cdot M' \cdot d\, l}{E\, I} \ldots\ldots (9)$$

Replacing the moments, M', in Equation (9) by only one moment of the value, 1, and by its proper reactions, and under the assumption that the supports are immovable in the direction of these reactions, so that the work of the reactions is nil, then, and then only, Equation (9) becomes

$$\alpha = \Sigma \frac{T_s \cdot T' \cdot d\, l}{E\, A} + \Sigma \frac{M_s \cdot M' \cdot d\, l}{E\, I} \ldots\ldots (10)$$

where T' and M' are the normal forces and moments produced in the structure by the moment, 1, and its reactions, which equation is identical with the author's Equation (6).

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Similarly, Equation (8) is transformed by the assumption of only one force, 1, acting on a structure with immovable supports in the direction of the reactions

$$\Delta = \Sigma \frac{T_s \cdot T' \cdot d\,l}{E\,A} + \Sigma \frac{M_s \cdot M' \cdot d\,l}{E\,I} \quad \ldots\ldots\ldots (11)$$

T' and M' being the normal forces and moments produced by the force, 1, and its reactions.

Castigliano has deduced, from the principle of virtual displacements, another rule for finding the displacement of any point of a structure. He replaced the set of forces in Equation (1) by another set of forces which varied from the first only in this respect, that all forces of the old set remained the same except that the force, P_n, acting in the point, n, the displacement of which is to be found, is changed by the infinitesimal quantity, δP_n. The interior forces will then be changed, also, by the infinitesimal values, δT_s and δM_s.

Provided the work of the reactions due to δP_n is nil, Equation (1) becomes

$$\Sigma P \cdot \Delta + \delta P_n \cdot \Delta_n = \Sigma (T_s + \delta T_s) \cdot \Delta\, d\,l + \Sigma (M_s + \delta M_s) \cdot \Delta\, d\,\alpha \quad (12)$$

Subtracting Equation (1) from Equation (12) and dividing by δP_n

$$\Delta_n = \Sigma \frac{\delta T_s}{\delta P_n} \cdot \Delta\, d\,l + \Sigma \frac{\delta M_s}{\delta P_n} \cdot \Delta\, d\,\alpha$$

Introducing again Equations (6) and (7)

$$\Delta_n = \Sigma \frac{\delta T_s}{\delta P_n} \cdot \frac{T_s \cdot d\,l}{E\,A} + \Sigma \frac{\delta M_s}{\delta P_n} \cdot \frac{M_s \cdot d\,l}{E\,I} \quad \ldots\ldots\ldots (13)$$

and, similarly, for moments,

$$\alpha = \Sigma \frac{\delta T_s}{\delta M_n} \cdot \frac{T_s \cdot d\,\alpha}{E\,A} + \Sigma \frac{\delta M_s}{\delta M_n} \cdot \frac{M_s \cdot d\,l}{E\,I} \quad \ldots\ldots\ldots (14)$$

These equations are practically identical with Equations (10) and (11), as will be shown later.

Evidently,

$$T_s \cdot \frac{\delta T_s}{\delta P_n} = \frac{\delta T_s^2}{2\,\delta P_n},$$

$$M_s \cdot \frac{\delta M_s}{\delta P_n} = \frac{\delta M_s^2}{2\,\delta P_n}$$

hence, Equations (13) and (14) become

$$\Delta_n = \frac{\delta \left(\Sigma \frac{T^2\, d\,l}{2\,E\,A} + \Sigma \frac{M^2\, d\,l}{2\,E\,I} \right)}{\delta P_n} \quad \ldots\ldots\ldots (15)$$

$$\alpha = \frac{\delta \left(\Sigma \frac{T^2 \cdot d\,l}{2\,E\,A} + \Sigma \frac{M^2 \cdot d\,l}{2\,E\,I} \right)}{\delta M_n} \quad \ldots\ldots\ldots (16)$$

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The expressions in the denominator are those of Clapeyron for the actual work performed by gradually applied forces acting on a structure in equilibrium, and Equations (13) to (16) simply state that the displacement or the rotation of a point, n, acted on by a force, P_n, or a moment, M_n, equals the first partial derivative of the expression for the work of the gradually produced interior forces, taken according to δP_n or δM_n.

At an immovable support, Δ or α must be nil, hence the first derivative must be zero. This can only happen for a condition where the work is a maximum or a minimum. It can be easily shown that the work cannot be a maximum, hence it must occur at a minimum. Therefore, Equations (13) to (16) have been called the principle of least work. If it is desired to find the displacement of any point at which no force happens to act, it is assumed that a fictive force or moment is acting at that point, P_n or M_n; the general expressions for the work performed by all the forces and moments and P_n and M_n are written down, the first derivative is found, and in this derivative and the general equation of the moment, P_n and M_n are assumed to vanish. The normal forces and moments produced in any structure by a single load or moment, being always linear functions of P_n or M_n, it follows that $\frac{\delta T}{\delta P_n}$ and $\frac{\delta M}{\delta M_n}$ are, in fact, the values of the normal forces and moments corresponding to $P_n = 1$ and $M_n = 1$.

In applying any of these methods to actual structures, great care must be taken first to transform an indeterminate structure into a so-called statically determinate system by the assumption of the proper reactions or forces; one must always consider that the reactions count just as heavily as any other force, and that any movement of the abutments will contribute to the work performed by the exterior forces and must appear on the left side of Equations (1) to (16). If, for example, one abutment has a displacement, e, in the direction opposite to the reaction, R, and a rotation, t, in the direction of the moment of the reaction, O, then there must be added to the left of:

Equation (1) $\quad - R \cdot e + O \cdot t$

(10) $\quad - R' \cdot e + O \cdot t$, when R', V, and O are the reactions due to $P' = 1$ and $M' = 1$,

(13) and (15) $\quad - \frac{\delta R}{\delta P_n} \cdot e + \frac{\delta O}{\delta P_n} \cdot t$

(14) " (16) $\quad - \frac{\delta R}{\delta M_n} \cdot e + \frac{\delta O}{\delta M_n} \cdot t$

To show that the theory of least work is practically identical with the author's principle, his Case II will be solved by that principle, and

it will be assumed that a moment, M_n, instead of a moment, 1, is acting at the right end of the beam; then, evidently, Mr. Mensch.

$$\text{for the left of } P \ldots M_s = P \cdot x \cdot \frac{(l-a)}{l} + M_n \cdot \frac{x}{l} \left\| \begin{matrix} \frac{\delta M_s}{\delta M_n} = \frac{x}{l} \\ \frac{\delta M_s}{\delta M_n} = \frac{x}{l} \end{matrix} \right\| \begin{matrix} M_n = 0 \text{ in } M_s \\ M_n = 0 \text{ " } M_s \end{matrix}$$

$$\text{" " right of } P \ldots M_s = P \cdot a \cdot \frac{l-x}{l} + M_n \cdot \frac{x}{l}$$

and this, introduced in Equation (14), gives

$$\alpha = \int_0^a \frac{x}{l} \cdot \frac{P\,x\,(l-a)}{l}\,dx + \int_a^l \frac{x}{l}\,P \cdot a \cdot \frac{l-x}{l}\,dx \ldots\ldots$$

which is identical with Professor Swain's Equation (9).

Nothing is saved in the integrations, but Professor Swain saves the writing down of two expressions affected by δM_n.

If it is desired to find the displacement and rotation of a point of a fixed arch, and if the same method is used as that shown in the author's Case II, it would lead to rather long formulas.

By a happier choice, of transforming the indeterminate structure into a statically determinate system, very much simpler expressions can be obtained. Fig. 27 shows how the arch may be transformed into a cantilever by assuming that, at the point, B, where the displacement is to be found, the right end of the arch is cut off, and at that section a horizontal force, a vertical force, and a moment, are applied to re-establish the equilibrium. In order to find the displacements by Professor Swain's principle, there is applied, besides, a horizontal and a vertical force, 1, at that section, and a moment, 1, and these new forces must be counterbalanced by the proper reactions at the left abutment.

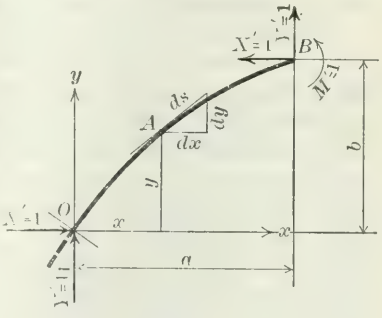

FIG. 27.

M_s, at any point, x, y, is again the moment of the forces acting on the structure the deflection of which it is desired to find. The indeterminate values of the reactions must duly appear in the expression for the moment.

T_s is the corresponding normal force.

$M' = 1$ will produce, in any point, x, y, a moment 1;

$X' = 1$ " " " " " " " " " " $(b - y)$ and a normal force $\left(1 \cdot \frac{dx}{ds}\right)$;

$Y' = 1$ will produce, in any point, x, y, a moment $(a - x)$ and a normal force $\left(-1 \cdot \frac{dy}{ds}\right)$;

Mr. Mensch.

hence, the change of angle of the arch between the point, O, and the point, B,

$$\alpha = \int_0^a \frac{M_s \cdot 1 \cdot ds}{EI} = \int_0^a \frac{M_s \cdot ds}{EI} \quad \ldots\ldots\ldots\ldots (17)$$

the horizontal displacement of the point, B, in the direction, $X' = 1$,

$$\Delta_x = \int_0^a \frac{T_s \cdot \frac{dx}{ds} \cdot ds}{EA} + \int_0^a \frac{M_s (b - y) \cdot ds}{EI}$$

$$= \int_0^a \frac{T_s \cdot dx}{EA} + b \int_0^a \frac{M_s \cdot ds}{EI} - \int_0^a \frac{M_s \cdot y \cdot ds}{EI} \quad \ldots (18)$$

the vertical displacement of the point, B, in the direction, $Y' = 1$;

$$\Delta y = \int_0^a \frac{T_s \cdot \left(-\frac{dy}{ds}\right) \cdot ds}{EA} + \int_0^a \frac{M_s \cdot (a - x) \cdot ds}{EI}$$

$$= -\int_0^a \frac{T_s \cdot dy}{EA} + a \int_0^a \frac{M_s \cdot ds}{EI} - \int_0^a \frac{M_s \cdot x \cdot ds}{EI} \quad \ldots (19)$$

which equations will be found useful in many arch problems.

The author's treatment of Cases VI and VII cannot be said to be new. The same cases and very many others are identically treated by Müller-Breslau,* and it is the writer's impression that the French engineers of the Paris Fair of 1878 designed the many arched structures of the exhibition buildings by this method; also that a description of the method appeared† at about that time. Müller-Breslau has also shown how to simplify greatly the calculations, especially in structures of a symmetrical shape. For example, he transforms the arch structure, shown in Fig. 28, into a kind of statically determined continuous structure in which the left abutment is only able to take vertical reactions of the same amount as those which would be taken by a common girder, and he connects the left end of the arch by an absolutely rigid body with the apex at O, at which point, in order to re-establish the equilibrium, he applies the horizontal thrust, X, the moment M_0, and the indeterminate portion of the vertical reaction, Y. He proves that the point, O, must be the center of gravity

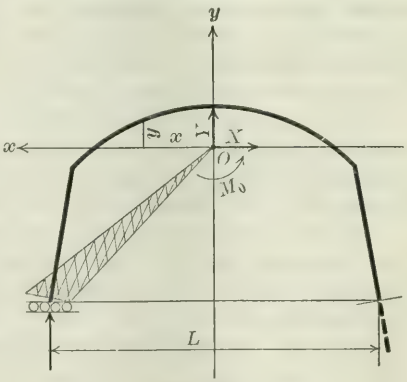

FIG. 28.

* "Graphic Statics of Indeterminate Structures", 1892.

† In *Le Génie Civil.*

of the so-called elastic weights, $\frac{d\,s}{I}$, of each point of the structure, and Mr. Mensch.
if this point is taken as the axis of a system of co-ordinates and M'' are the moments of a statically determined system which rests on rollers, and on which no indeterminate reactions are acting, in fact, when M'' is the common moment of a common beam, then,

$$X = \frac{\int_0^l \frac{M'' \,.\, x \,.\, d\,s}{E\,I}}{\int_0^l \frac{x^2 \,.\, d\,s}{E\,I}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$Y = \frac{\int_0^l \frac{M'' \,.\, y \,.\, d\,s}{E\,I}}{\int_0^l \frac{y^2 \,.\, d\,s}{E\,I}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$M_0 = \frac{\int_0^l \frac{M'' \,.\, d\,s}{E\,I}}{\int_0^l \frac{d\,s}{E\,I}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

which are, of course, very much simpler expressions than the often extremely sensitive, and always very involved, Equations (19), (20), and (21), of the author.

French engineers, for nearly 40 years, have been very successful in finding still more important short-cuts in the design of indeterminate structures, and the writer confesses that he would have never designed and built indeterminate structures if these short-cuts, and others which he developed himself, had not reduced the labor of computation often to 10% and sometimes even to 1% of that required according to the methods shown in the ordinary textbooks.

To compute the simplest arch by the ordinary methods in vogue is a considerable task, if the change of the moment of inertia, the change of temperature, and the shortening of the arch are duly taken into consideration. If, in addition, the displacement and rotation of points are to be found, in order to decide about expansion joints in reinforced concrete bridges, it will take the best engineer, if he is not an expert in arch design, probably many weeks of steady and hard work to design a bridge. Generally, the work is done very roughly, temperature and shortening stresses are neglected, and to find displacements of points is not dreamed of. This explains the fact that many bridges crack, although designed with a large factor of safety, also the fact that most American arched bridges, on account of the uncertainty of the designer, are in general very much heavier than those of European design.

Mr. Mensch.

It is very important to know that the application of the principles as here developed have many limitations, *viz.:*

(1) The structure must be in a perfect state of equilibrium.
(2) The deformations must be extremely small.
(3) The stresses must be within the elastic limit, and must follow Hooke's law.
(4) There must be absence of all vibration.
(5) It is assumed that all the work performed by the exterior forces is consumed by the work in deforming the structure, and that the production of heat, and chemical and crystallographic changes are neglected.
(6) Changes of temperature must be computed separately.
(7) Secondary stresses are neglected.
(8) The work in overcoming friction at the abutments or joints is neglected.
(9) Poisson's ratio is neglected.
(10) The influence of the curvature of the structure is neglected.
(11) It is not permitted to apply indiscriminately the statics of a rigid body. In a framed structure, for example, where a force is acting at a lower joint, it is not permitted to assume that its point of application might be indifferently at any point of its line of action. If the line is extended to the upper chord, it would probably produce bending there, and the work done would be quite different.
(12) The influence of shearing forces (of great importance in dams, for example), is neglected.

These limitations apply to both determinate and indeterminate systems, and have as yet received comparatively little study by investigators. Some of these points, however, are of great importance, and the figures may be out 50% and more; and in many structures one cannot predict the factor of safety.

Of the utmost importance is the behavior of materials when it differs from Hooke's law, as when a structure is stressed beyond the elastic limit, or, as in concrete construction, where the time-flow or the plasticity of the concrete might reduce the modulus of elasticity for the dead-load stresses to perhaps one-tenth of the value of that under a short-time load. As a rule, the factor of safety is increased, due to the flow of the material, and the stresses in the dangerous section remain nearly constant with gradually increased loads, and the comparatively stronger parts take up more stresses than have been computed until a new state of equilibrium is reached.

Mr. Turneaure.

F. E. TURNEAURE,* M. AM. SOC. C. E. (by letter).—This paper presents a very interesting development of a principle in the theory of

* Madison, Wis.

Mr. Turneaure.

structures which, so far as the writer knows, has not been made use of heretofore. The principle itself is not exactly new to the writer, and its application to the determination of stresses in redundant members, or the determination of redundant reactions, is quite fully set forth in his revision of Johnson's "Framed Structures", Part II, 1910. Chapter VI of this work reviews, in the first few pages, the general principles and formulas used in the determination of redundant stresses, and then applies these principles to quite a variety of selected problems. In the general formula, Equation (5), page 335, of this work, an equation is given expressing the relation between redundant stresses, S_1, S_2, S_3, etc., and the stresses and bending moments due to the external loads. Following this equation, the statement is made:

"Instead of assuming as the unknowns the various direct stresses, S_1, S_2, etc., it is sometimes more convenient to take certain of the bending moments as the unknowns, or certain direct stresses together with certain moments. In this case the values of u_1 and m_1, corresponding to an unknown bending moment, M_1, are respectively the stresses and moments in the various members due to a bending moment of unity in the redundant member. The resulting equations are then the same as Equation (5) excepting that certain unknown moments, M_1, M_2, etc., are used instead of the stresses, S_1, S_2, etc."

The general method thus fully set forth, applied to the problem given by Professor Swain as Case VI, would, on substitution of the proper notation in Equation (5) referred to, produce precisely his Equations (16), (17), and (18). In presenting this method of attack, it was also recognized by the writer, as indicated in the subsequent article, especially on page 336, that the quantities, $\Sigma \frac{s\,l}{E\,A} \cdot u_1$ and $\Sigma \int \frac{M\,d\,x}{E\,I} \cdot m_1$ (identical with those of Professor Swain's Equation (4)), represent the change in angle of the member under the actual conditions of loading, the quantities, u_1 and m_1, being the direct stress and bending moment, respectively, in any member due to a unit moment applied at the point the angular change of which is desired.

While thus setting forth the practicability of applying unit bending moments as well as unit forces in problems of this kind, the writer, as a matter of fact, did not follow this method in numerical problems, primarily for the reason that the solution adopted in any particular case was that which appeared to be the most convenient and expeditious. For example, in the solution of various problems of portals or quadrangular frames fixed at the base, the method for arches was generally used. This method assumed as the redundant quantities the shear, thrust, and bending moment at the crown. Three equations for the determination of these three unknown quantities were derived by equating the horizontal and vertical components of deflection and the angular change at this point for the two sides of the arch. The expres-

Mr. Turneaure.

sion for the angular change of the arch axis at the crown is really identical with the form of expression derived by Professor Swain, although it is apparent that a mere familiarity with this expression would not be apt to lead to the generalization in Professor Swain's paper. His explanation of how this principle may be applied in determining changes of inclinations of any member of a structure is very interesting, and will doubtless prove of value in many practical problems.

Referring to the foot-note on page 622, in which Professor Swain explains the character of the units in Equation (1), it seems to the writer to be somewhat more logical to consider that the quantity, t, is merely a ratio, or an abstract number, which, multiplied by $\frac{s\,l}{E\,A}$, results in a length. The quantity, $\frac{s\,l}{E\,A}$, is the actual change in length or movement of the member distorted, and the quantity, t, is simply a factor by which this change in length is multiplied in order to determine the contribution of this particular member to the deflection desired. It is a mere ratio, and when determined by the ordinary process of calculating the stress in a member due to a unit load, it represents, not the stress itself, but the ratio of stress to load; but, the load happening to be unity, the numerical value of the ratio is the same as that of the stress. This factor or ratio, t, can be determined in other ways, as, for example, by the principle of virtual velocities, or by the principle of work, or by applying differential calculus to the linear dimensions of the framework. In any case, it may be looked on as simply a ratio, and hence the product of this ratio multiplied by the length is a length.

Mr. Maney.

G. A. Maney,* Esq. (by letter).—The writer submits the following discussion on the "slope-deflection" method† as applied to frame analysis:

A difficulty in the analysis of frames for concrete ships is found in the irregularity in their make-up and the inapplicability of ordinary methods to secure accurate results.

The transverse framing of a concrete ship consists generally of a large number of members, of varying stiffness, connected by rigid joints, and continuous throughout. Many of these members have great variation in stiffness throughout their length, others are curved, and

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† The so-called slope-deflection method, as first treated by the writer, was published in "Studies in Engineering, No. 1" of the University of Minnesota, in 1915. A special application of this method was made shortly afterward in *Bulletin No. 80*, of the University of Illinois Engineering Experiment Station, in a paper on "Wind Stresses in Office Buildings." On p. 412 of the "Concrete Engineers' Handbook", by Messrs. Hool and Johnson, credit is given to the writer for the method for the general treatment of rigid frames, which is used in Chapter 10, on "Moments in Rigid Building Frames."

still others are arches in their mode of action, and vary as to loading and end conditions. Mr. Maney.

Perhaps the most unusual problem is that of a rigid frame having one or more of its members as irregular arches, with ends neither fixed nor free, but subjected to a degree of restraint determined entirely by the relative stiffness and loading of all the members of the structure.

In treating this problem, the method developed by the writer is similar to that described by Professor Swain in that the tangent direction or slope of the elastic curve of the member, at certain critical points, is used as a medium.

Discussion and Derivation of Method.—The end conditions of any member subjected to any possible distortion (in one plane) due to flexure, may be expressed or defined by certain functions of the elastic curve which will be called S_A, S_B, R_1, and R_2. Each of these quantities, as used in the calculations, is some multiple of the tangent

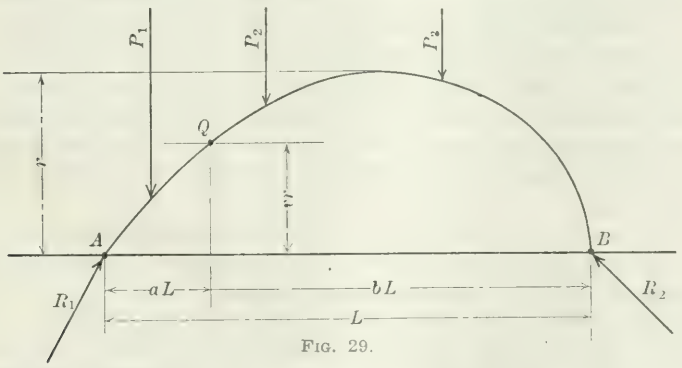

FIG. 29.

values of small angles. The angles, in turn (or their tangents), are measured by the quotients of certain deflections divided by distances on the member in question.

Any condition of distortion, as determined by these end conditions in combination with certain loads applied between the ends of the member, can be effected by the application of certain end moments, M_A and M_B, together with a force, F, at one end, applied in the direction of the line, AB, connecting the ends. The general expression for the bending moment, M, at any section, for the most general case of a curved member with flexure in the plane of the member only, takes the following form:

$$M = a\,M_B + b\,M_A + v\,F\,r + M' \qquad (1)$$

The quantities, a, b, r, v, and M' will be defined with the aid of Fig. 29.

Mr. Maney.

a is a ratio of the distance from A to the projection of any point, Q, on the line, AB, connecting the ends of the member, divided by the length of AB;

b is this ratio where the distance to the projection of any point is measured from B;

r is the rise, or distance, measured at right angles, from AB to the point on the axis of a curved member which is farthest from AB;

v is the ratio of this same distance for any other point to the distance, r;

M' is the bending moment at any point which is calculated on any statically determinate basis.

In general, M' is assumed to be zero at the ends of the member. Then, any direction of one reaction (as R_1) is assumed, and its value is calculated by taking moments of the forces about B and equating to zero. R_2 may then be determined, both as to value and direction. The member now being in equilibrium under applied loads, the value of M' for all the points along the axis (ten are generally sufficient) may be calculated. It becomes evident that any set of M' values consistent with the applied loads may be used.

In order to make these end conditions of the elastic curve of the general member the unknowns to be solved for, rather than the unknown bending moments and thrusts at the ends, it becomes necessary to express the latter (M_A, M_B, and $F\,r$) in terms of the former (S_A, S_B R_v, and R_H).

This can be done because of the definite and well-recognized relations between the extreme fiber deformations of a member subjected to bending, and the accompanying deflections.

Before this is done, a more definite statement concerning S_A, S_B, R_v, and R_H, will be made. These quantities are all ratios which, when multiplied by L (for S_A, S_B, and R_v) or by r (for R_H), will give certain multiples of deflection values. The deflections, $S_A\,L$, $S_B\,L$, and $R_v\,L$, are all measured in a direction at right angles to the line, $A\,B$.

The deflection of the end, A, from a tangent to the elastic curve at B, and in a direction at right angles to $A\,B$, is equal to $R_v\,L$, or $S_B\,L$, or a combination of the two, depending on the final direction of the line between the ends relative to its initial direction. In the general formula, this deflection must be made a function of both R_v and S_B. Also, the deflection of the end, B, from a tangent to the elastic curve at the end, A, will be a function of both R_v and S_A. The deflection of either end with respect to the other, which takes place in a line between these, is measured by the quantity, R_H.

To the foregoing deflections, are equated their values, in terms of the bending moment at any section. As before indicated, this moment value will be a function of the unknown moments, M_A, M_B, and $F\,r$,

which are the values to be found as a result of a solution by this method. Mr. Maney.

The member under consideration will be divided into ten or more sections, having no necessary relation to each other, as far as relative length and stiffness is concerned. The value of $\frac{l}{I}$ for each section is obtained, l being the axial length of the section and I the average moment of inertia. The calculation of the values of $\frac{l}{I}$ is simplified by the consideration that only relative values of these quantities need be used.

By means of the relation between the deformation of the extreme fibers in a bending member and its deflections, the value of these deflections will be expressed in terms of the bending moments which determine extreme fiber unit deformations.

Consider any small length, l, along the axis of any member subjected to bending stress, over which M is the average bending moment and I is the average moment of inertia. Applying the fundamentals of deflection, it will be found that $\sum_{B}^{A} b\, L \left(\frac{M\, l}{E\, I}\right)$ equals the deflection of the end, B, from a tangent to the member at A, and in a direction at right angles to $A\ B$. Therefore:

$$b\, L \left(\frac{M\, l}{E\, I}\right) = R_b\, L - S_A\, L.$$

$$\text{Likewise } a\, L \left(\frac{M\, l}{E\, I}\right) = S_B\, L - R_b\, L;$$

$$\text{and } r\, r \left(\frac{M\, l}{E\, I}\right) = R_H\, r.$$

Then, neglecting the value of E, which is common to all the equations, and knowing that only relative values of $\frac{l}{I}$ are necessary, the simpler form of these three fundamental equations of deflection are obtained:

$$b\, M \left(\frac{l}{I}\right) = R_b - S_A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$a\, M \left(\frac{l}{I}\right) = S_B - R_b \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$r\, M \left(\frac{l}{I}\right) = R_H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In these equations it is necessary to govern the signs of the $\left[b\, M \frac{l}{I} \text{ (etc.)}\right]$ quantities by one method, and the signs of S and R by an entirely different scheme.

Mr. Maney.

Also, the general frame problem must be divided into two entirely different phases. In one of these the object is to determine the effect of end conditions and loads on a curved member of varying stiffness, with regard to the distribution of moments throughout the length of the member. The common problem of the hingeless reinforced concrete arch is one of the simple forms of this problem, where the quantities, S and R, which determine the end conditions, are zero. In a problem of this sort, the bending moment at any section of a member will be considered as plus or minus, according as it produces tension on the lower or upper extreme fiber. Lower or upper extreme fibers may be determined entirely by a separate sketch of the member in any position, without relation to its exact relative position or orientation in the structure.

After finding the effect of all possible end conditions and applied loads on the end moments of this member, M_A and M_B, the sign of each of these moments is determined by its effect on the rotation of the joint in the structure to which it is adjacent. If it tends to rotate the joint clockwise it will be given a plus sign, and if counter-clockwise, a minus sign.

In any straight member, an individual solution to find the effect of applied loads and end conditions is not necessary. The moment at any end, A, of a straight member, AB, with uniform stiffness may be expressed in the general form:

$$M_A = M_{F_A} + \frac{2EI}{L}(3R_v - 2S_A - S_B) \quad \ldots\ldots\ldots (5)$$

M_{F_A} is the fixed beam moment at A which gives the effect of applied loads when the end conditions are zero. This case is that of a fixed beam for a straight member, or a hingeless arch for a curved member.

For any end conditions other than those expressed by zero values of R and S, the effect of end slope values, S_A and S_B, and the relative end deflection, R_v, on the final moment, M_A, may be obtained by a direct algebraic addition of the end moments due to these quantities, to M_F.

It should be noted that the values of these end conditions, S and R, are determined entirely with reference to the original direction of the line connecting the ends of the member in its unstrained conditions.

Also, a clockwise rotation of the end slope values of the elastic line will make S_A and S_B plus, and a counter-clockwise rotation, minus. Similarly, if either end rotates in a clockwise direction relative to the other end, R_v will be plus.

When the general expression for M, as given by Equation (1), is substituted in Equations (2), (3), and (4), the following equations are obtained, and these, when solved simultaneously, will give perfectly general values for M_A and M_B, as well as Fr. This last quantity measures the effect of end thrust in the direction, AB, on all the

moments, and it will readily be seen that in any straight member this will not affect the moments in any primary sense: Mr. Maney.

$$\Sigma\, b \frac{l}{I}(b\,M_A + a\,M_B + r\,F\,r) = -S_A + R_v - \Sigma\, b \frac{l}{I} M'$$

$$\Sigma\, a \frac{l}{I}(b\,M_A + a\,M_B + r\,F\,r) = -R_v + S_B - \Sigma\, a \frac{l}{I} M'$$

$$\Sigma\, r \frac{l}{I}(b\,M_A + a\,M_B + r\,F\,r) = +R_H \qquad -\Sigma\, r \frac{l}{I} M'$$

These three equations are solved on page 670.

The following application of this method to a typical ship-frame problem has been worked out by R. A. Cummings, Jr., Jun. Am. Soc. C. E. This problem considers the member around the bilge as an arch, with its end tangents free to turn a certain distance depending on the stiffness of the other members framing into it, as well as on the applied loads. It was simplified by the assumption of uniform stiffness throughout the length of the arch axis, and also by the fact that, although the ends here are free to twist, they will not have relative motion. These facts eliminate $\frac{l}{I}$ as a factor in the arch solution, and also eliminate R as an unknown to be solved for in the simultaneous equations. The four unknowns, S_B, S_C, S_D, and S_E, are obtained by the solution of four simultaneous equations for the joints of the frame, B, C, D, and E, respectively. Each of these equations expresses the statically determinate condition, that the summation of the tendencies of all the moments acting on a joint must equal zero.

Fig. 30 shows a diagram indicating the clear or effective span of the members, and Table 3 gives the calculation of the end bending moments of all the members for the condition where all the values of S are zero, which is the fixed-beam moment for the straight members and the hingeless-arch end moments for the arch member, AB.

In Fig. 30, for the lengths of members take the clear span and not the distance from center to center of the supports.

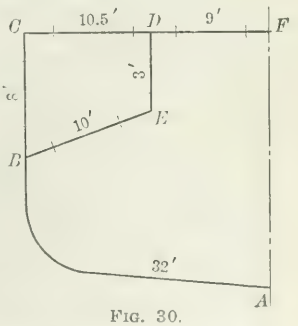

FIG. 30.

Find $\frac{I}{l}$ for all members, and express K in terms of AB, equal to 10.

$$\frac{I}{l} = \frac{d^3}{l} = K$$

$K_{AB} = 10$ $\qquad$ $K_{CD} = 3$

$K_{BC} = 4$ $\qquad$ $K_{DE} = 10$

$K_{BE} = 3$ $\qquad$ $K_{DF} = 4$

Solving these three equations simultaneously gives:

$$M_A = -\frac{K_1 S_A - K_2 S_B + (K_1 + K_2) R_v - K_3 R_H + K_2 \Sigma a \frac{l}{I} M' - K_1 \Sigma b \frac{l}{I} M' + K_3 \Sigma v \frac{l}{I} M'}{K_1 \Sigma b^2 \frac{l}{I} - K_2 \Sigma a b \frac{l}{I} - K_3 \Sigma b v \frac{l}{I}}$$

$$M_B = +\frac{K_2 S_A + K_4 S_B - (K_2 + K_4) R_v - K_5 R_H - K_4 \Sigma a \frac{l}{I} M' + K_2 \Sigma b \frac{l}{I} M' + K_5 \Sigma v \frac{l}{I} M'}{\text{(Same denominator as for } M_A)}$$

$$Fr = +\frac{K_3 S_A - K_5 S_B - (K_3 - K_5) R_v + K_6 R_H + K_5 \Sigma a \frac{l}{I} M' + K_3 \Sigma b \frac{l}{I} M' - K_6 \Sigma v \frac{l}{I} M'}{\text{(Same denominator as for } M_A)}$$

$$K_1 = \left(a^2 \frac{l}{I}\right)\left(v^2 \frac{l}{I}\right) - \left(a v \frac{l}{I}\right)^2 \qquad K_4 = \left(b^2 \frac{l}{I}\right)\left(v^2 \frac{l}{I}\right) - \left(b v \frac{l}{I}\right)^2$$

$$K_2 = \left(a b \frac{l}{I}\right)\left(v^2 \frac{l}{I}\right) - \left(a v \frac{l}{I}\right)\left(b v \frac{l}{I}\right) \qquad K_5 = \left(b^2 \frac{l}{I}\right)\left(a v \frac{l}{I}\right) - \left(a b \frac{l}{I}\right)\left(b v \frac{l}{I}\right)$$

$$K_3 = \left(a^2 \frac{l}{I}\right)\left(b v \frac{l}{I}\right) - \left(a b \frac{l}{I}\right)\left(a v \frac{l}{I}\right) \qquad K_6 = \left(a^2 \frac{l}{I}\right)\left(b^2 \frac{l}{I}\right) - \left(a b \frac{l}{I}\right)^2$$

(NOTE.—All quantities, such as $\left(a^2 \frac{l}{I}\right)$, etc., mean $\Sigma a^2 \frac{l}{I}$.)

These summations may be obtained conveniently by the tabulated form indicated in the problem, and, in any case, are known functions of the shape (a, b, and v), the distribution of stiffnesses, $\frac{l}{I}$, and the loading, M'.

Mr. Maney.

Mr. Maney.

Table 4 contains the complete general expression for the bending moment at each end of every member in the frame. Each of these moments takes the form

$$M_{1-2} = M_F + K\,(3\,R - 2\,S_1 - S_2).$$

M_F, the fixed-beam moment for zero change in end tangent direction, has been calculated, and R has a zero value in these equations, because the action of the frame admits of no relative displacement of the joints. K is a measure of the relative stiffnesses of the various members.

In the member, BA, the end moment, M_{BA}, with zero change of slope, has an arch moment of — 116 000 in-lb. This was calculated in the example under arches. The arch moment sign, as calculated, is negative, which means that the outside arch fibers are tensile. To cause tension on the outside, that portion of the arch near the abutment, B, must rotate counter-clockwise, hence the negative slope deflection sign.

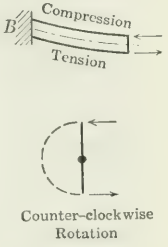

TABLE 3.—Calculation of Fixed-Beam End Moments.

Point.	Member.	Loading.	Sign.	Moment.	Moment value, in inch-pounds.
A	AB		+	From Previous Arch Solution.	3 527 000
B	BA			" " " "	116 000
B	BC		+	$M = \frac{W\,l}{10} = \frac{15\,600 \times 8}{10} \times 12$	150 000
B	BE	None			
C	CB		—	$M = \frac{W\,l}{15} = \frac{15\,600 \times 8 \times 12}{15}$	100 000
C	CD		+	$M = \frac{w\,l^2}{12} = \frac{2\,000 \times 10.5 \times 10.5 \times 12}{12}$	220 000
D	DC		—	" " "	220 000
D	DE	None			
D	DF		+	$M = \frac{w\,l^2}{12} = \frac{2\,000 \times 9 \times 9 \times 12}{12}$	162 000
E	EB	None			
E	ED	None			

Mr. Maney.

TABLE 4.—Stiff Frame Containing an Arch Member.

Joint.	Member Equation.
B	$M_{BA} = -116\,000 - 10\,(4.50\,S_B - 0.71\,S_A)$. S_A = zero.
	$M_{BC} = +150\,000 + 4\,(-2\,S_B - S_C)$
	$M_{BE} = 0 + 3\,(-2\,S_B - S_E)$
C	$M_{CB} = -100\,000 + 4\,(-2\,S_C - S_B)$
	$M_{CD} = +220\,000 + 3\,(-2\,S_C - S_D)$
D	$M_{DC} = -220\,000 + 3\,(-2\,S_D - S_C)$
	$M_{DF} = +162\,000 + 4\,(-2\,S_D - S_F)$. S_F = zero.
	$M_{DE} = 0 + 10\,(-2\,S_D - S_E)$
E	$M_{ED} = 0 + 10\,(-2\,S_E - S_D)$
	$M_{EB} = 0 + 3\,(-2\,S_E - S_B)$

TABLE 5.—Stiff Frame Containing an Arch Member.

Sum up All End Moments Around each Joint and Arrange as Follows, Solving the Simultaneous Equations by Elimination:

	S_B	S_C	S_D	S_E	Constant 1 000 in-lb.
Joint B	+ 59	+ 4	.....	+ 3	+ 34
" C	+ 4	+ 14	+ 3	.....	+ 120
" D	....	+ 3	+ 34	+ 10	− 58
" E	+ 3	.....	+ 10	+ 26	0
①′	+ 1	+ 0.008	.....	+ 0.051	+ 0.576
②′	− 1	− 3.500	− 0.750	.....	− 30.000
④′	− 1	.....	− 3.333	8.667	0
1′ + 2′ or ⑤′	....	− 3.432	− 0.750	+ 0.051	− 29.424
1′ + 4′ or ⑥′	....	+ 0.068	− 3.333	− 8.616	+ 0.576
⑤′	....	− 1.000	− 0.218	+ 0.015	− 8.573
⑥′	....	+ 1.000	− 49.015	− 126.706	+ 8.471
③′	....	+ 1.000	+ 11.333	+ 3.333	− 19.333
5′ + 6′ or ⑦′	....	.....	− 49.233	− 126.691	− 0.102
5′ + 3′ or ⑧′	...	.....	+ 11.115	+ 3.348	− 27.906
⑦′	....	.....	− 1.000	− 2.573	− 0.003
⑧′	....	.....	+ 1.000	+ 0.301	− 2.510
⑦′ + ⑧′ or ⑨′	....	.....	.....	− 2.272	− 2.513
⑨′	....	....	.....	+ 1.000	+ 1.106
⑧′	....	.....	+ 1.000	(+0.301)(+1.106)	− 2.510
⑤′	....	− 1.000	(−0.218)(−2.843)	(+0.015)(+1.106)	− 8.573
①′	+ 1.000	(+0.068)(+9.210)	.....	(+0.051)(+1.106)	+ 0.576

$S_E = +1.106$
$S_D = -2.843$
$S_C = +9.210$
$S_B = -0.106$

Mr. Maney.

The four simultaneous equations necessary for a solution of a stiff frame containing an arch member are indicated in Table 5. Each of these equations states that the summation of the moments at the ends of all members framing into a joint, which occur at the joint, should equal zero. There are as many unknowns to be found (S_B, S_C, S_D, and S_E) as there are equations.

The coefficients of these unknowns, with their signs changed due to transposition, are indicated opposite the equations at the left of the rows and below the unknown S values indicated at the tops of the columns. The details of the solution and the calculations for the S values are shown in Table 5.

Table 6 contains the calculations for the final values of the moments at the ends of the members. The values of S just found are substituted in the equations shown in Table 4. These final moment values are shown in Table 6 to sum up to zero at the various joints.

TABLE 6.—STIFF FRAME CONTAINING AN ARCH MEMBER.

Final Values of End Moments at Joints.

If $S_B = -0.106$ $S_D = -2.843$
$S_C = +9.210$ $S_E = +1.106$

Then:

Joint.	Member Equation.	Moment. 1 000 in-lb.
B	$M_{BA} = -116.0 - 10\,[(4.50)(-0.106)]$	$-$ 111.2
	$M_{BC} = +150.0 + 4\,[-2\,(-0.106) - (+9.210)]$	$+$ 111.0
	$M_{BE} = 0 + 3\,[-2\,(-0.106) - (+1.106)]$	$+$ 2.7
C	$M_{CB} = -100.0 + 4\,[-2\,(+9.210) - (-1.106)]$	173.3
	$M_{CD} = +220.0 + 3\,[-2\,(+9.210) - (-2.843)]$	$-$ 173.3
D	$M_{DC} = -220.0 + 3\,[-2\,(-2.843) - (+9.210)]$	230.6
	$M_{DF} = +162.0 + 4\,[-2\,(-2.843) - 0]$	$+$ 184.8
	$M_{DE} = 0 + 10\,[-2\,(-2.843) - (+1.106)]$	$-$ 45.8
E	$M_{ED} = 0 + 10\,[-2\,(+1.106) - (-2.843)]$	$+$ 6.3
	$M_{EB} = 0 + 3\,[-2\,(+1.106) - (-0.106)]$	$-$ 6.3
F	$M_{AB} = +3\,527 - 10\,[0.77\,(-0.106) + 0]$	$-$ 3527.8

On Fig. 31 the load line of the arch is shown as a part of two force polygons. The first has a pole which is arbitrary and is determined by any statically determinate condition where the end forces or rays pass through the ends of the axis. The second or correct pole,

Mr. Maney.

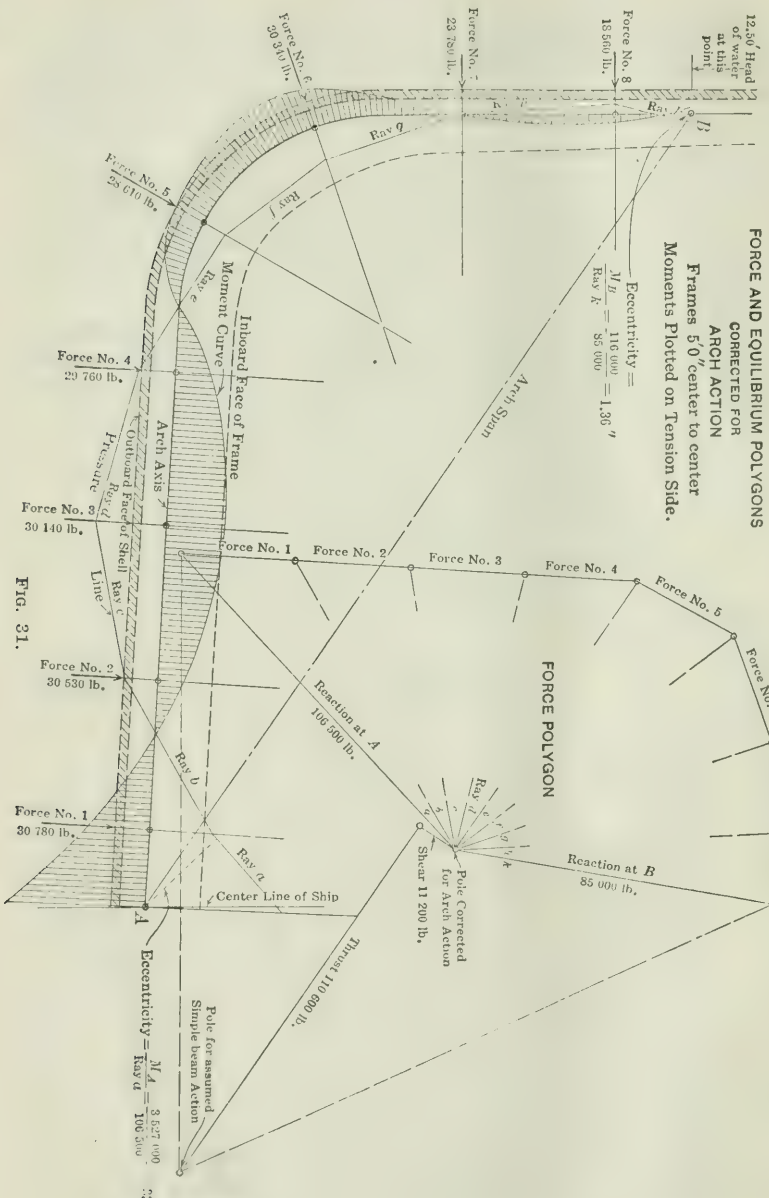
FORCE AND EQUILIBRIUM POLYGONS
CORRECTED FOR
ARCH ACTION
Frames 5′0″ center to center
Moments Plotted on Tension Side.
12.50′ Head of water at this point
Force No. 8 18 560 lb.
Force No. 7 29 780 lb.
Force No. 6 30 340 lb.
Force No. 5 25 670 lb.
Force No. 4 29 760 lb.
Force No. 3 30 140 lb.
Force No. 2 30 530 lb.
Force No. 1 30 780 lb.
Ray q
Ray p
Ray e
Ray d
Ray c
Ray b
Ray a
Eccentricity = M_B / Ray k = 116 000 / 85 000 = 1.36″
Arch Span
Inboard Face of Frame
Moment Curve
Arch Axis
Outboard Face of Shell
Pressure Line
Center Line of Ship
Reaction at A 106 500 lb.
Reaction at B 85 000 lb.
Shear 11 200 lb.
Thrust 110 000 lb.
Pole Corrected for Arch Action
Pole for assumed Simple beam Action
FORCE POLYGON
Force No. 1
Force No. 2
Force No. 3
Force No. 4
Force No. 5
Force No. 6
Force No. 7
Force No. 8
Eccentricity = M_A / Ray a = 3 627 000 / 106 500 = 3.12″

FIG. 31.

which determines the final forces acting on the arch, is found by correcting for M_A, M_B, and Fr. The final pressure line may then be drawn, which will check the calculated final moments on the arch member, AB. Mr. Maney.

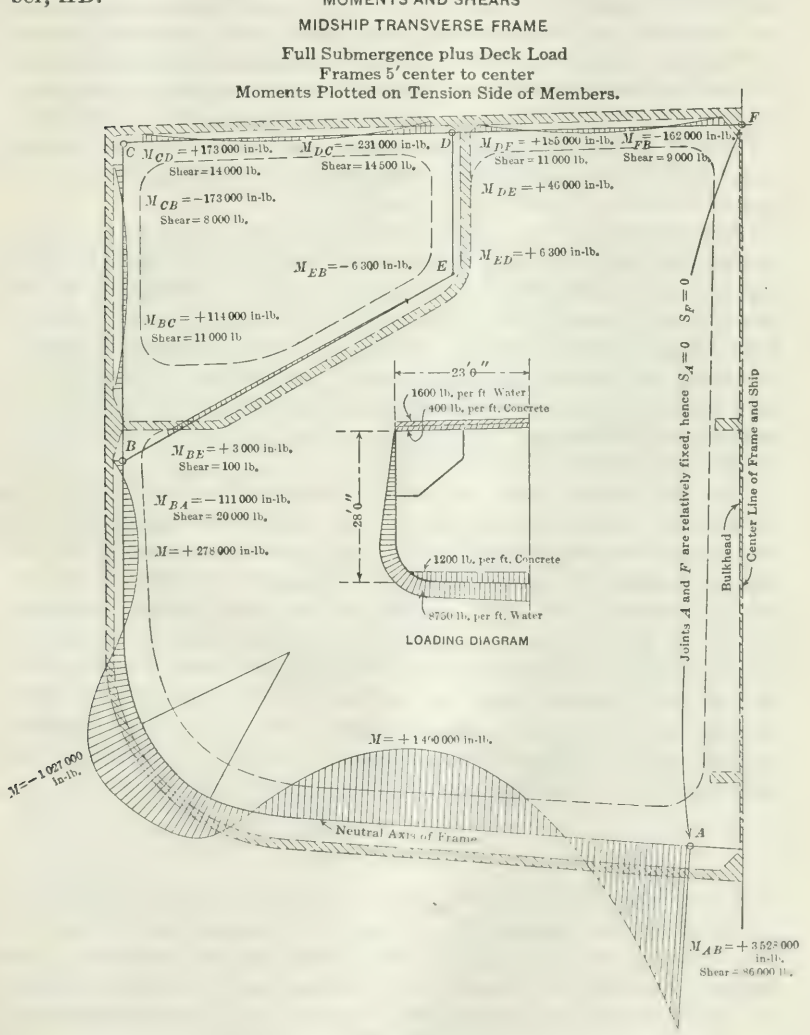
MOMENTS AND SHEARS
MIDSHIP TRANSVERSE FRAME
Full Submergence plus Deck Load
Frames 5′ center to center
Moments Plotted on Tension Side of Members.
C
D
E
F
B
A
M_{CD} = +173 000 in-lb.
Shear = 14 000 lb.
M_{DC} = − 231 000 in-lb.
Shear = 14 500 lb.
M_{DF} = +185 000 in-lb.
Shear = 11 000 lb.
M_{FB} = −162 000 in-lb.
Shear = 9 000 lb.
M_{CB} = −173 000 in-lb.
Shear = 8 000 lb.
M_{DE} = +46 000 in-lb.
M_{EB} = − 6 300 in-lb.
M_{ED} = + 6 300 in-lb.
M_{BC} = +114 000 in-lb.
Shear = 11 000 lb
23′ 0″
1600 lb. per ft. Water
400 lb. per ft. Concrete
28′ 0″
M_{BE} = + 3 000 in-lb.
Shear = 100 lb.
M_{BA} = −111 000 in-lb.
Shear = 20 000 lb.
M = + 278 000 in-lb.
1200 lb. per ft. Concrete
8750 lb. per ft. Water
LOADING DIAGRAM
Joints A and F are relatively fixed, hence S_A = 0 S_F = 0
Bulkhead
Center Line of Frame and Ship
M = + 1 470 000 in-lb.
M = −1 027 000 in-lb.
Neutral Axis of Frame
M_{AB} = + 352 000 in-lb.
Shear = 86 000 lb.

FIG. 32.

Fig. 32 contains sketches indicating the loading for the whole frame, and the final moments throughout the frame, plotted on the tension side of the axis of the members. The shear values are also indicated.

Mr. Burr.

WILLIAM H. BURR,* M. AM. SOC. C. E. (by letter).—This interesting paper is devoted to the establishment and application of what is stated to be a new principle in the determination of stresses and strains in that class of structures in which the axes of members are subject to angular displacement due to stresses of flexure or bending, as arched ribs or simple beams, and trusses with members subjected to bending as well as direct stresses. If any new principle leading to simpler forms of treatment for such structures can be established, it certainly will be welcomed. This discussion is not intended to be a detailed consideration of the subject matter of Professor Swain's paper, but rather to make some simple suggestions regarding the fundamental features of the author's analysis.

Apparently the basic idea on which the new principle is founded is that involved in the establishment of Equations (3) and (4) of the paper. It is to be observed that Equation (3) is established by first using a unit moment, that is, by applying a couple to the given section the moment of which is unity. Under the action of that unit moment, the stress of tension or compression, r, is produced. Equation (3) then legitimately follows from the preceding unnumbered equation, by introducing "(change of length)" due to the stress or stresses produced by the actual loads on the structure.

Let the case of beams or members subjected to bending only be considered. The second member of the author's Equation (3), with the summation sign removed, is to be applied in succession to each section throughout the entire beam, making it necessary to suppose the application of the unit moment to each such section, otherwise, the proper proportionate relation between r and "(change of length)" would not exist. On page 625 of the paper, however, a value of r is written, which results from the application of a moment, M_r, which, in general, has a value quite different from unity. The resulting expression for the angular displacement is Equation (1), which is the second term of the second member of the author's Equation (4):

$$\alpha = \sum \frac{M_s M_r \, d\, l}{E\, I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

It is clear, therefore, that the analytical error due to using the stress, r, produced by the moment, M_r, instead of a moment unity, has been introduced in the analysis. In other words, the M_r appearing in the author's Equation (4) should be considered unity, making Equation (1) identical with the old standard formula for angular displacement, which will be recognized at once in Equation (2):

$$\alpha = \sum \frac{M \, \Delta \, l}{E\, I}; \text{ or } \int \frac{M \, d\, x}{E\, I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* New York City.

Mr. Burr.

The results of this analytic error can be pointedly shown by the following two illustrations. In Equation (9), of the paper, the author makes an application of the "new principle" to a beam simply supported at each end and carrying a load at any point, the expression for the angular rotation or displacement being taken at the right-hand end of the beam. The moment, M_r, varies uniformly from unity at the right-hand end of the beam to 0 at the left-hand end. If the load, P, is placed at the center of the span, the angular displacement at the right-hand end of the beam becomes:

$$\alpha_1 = \frac{P\,l^2}{16\,E\,I} = \frac{\frac{1}{2}\,P\left(\frac{l}{2}\right)^2}{2\,E\,I} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Each half of such a beam is a cantilever, loaded at its free end with $\frac{1}{2}\,P$, the fixed end of the cantilever being at the center of span of the beam. The angular displacement of the free end of such a cantilever in reference to its fixed end may be found by taking one-half of α_1, given by Equation (3), for the reason that the summation or integration shown by Equation (3) covers the entire beam, the length of which is twice that of the cantilever. This will make the angular rotation for the free end of the cantilever shown by Equation (4), which is one-half only of the correct value shown by the author's Equation (7):

$$\alpha = \frac{\alpha_1}{2} = \frac{\frac{1}{2}\,P\left(\frac{l}{2}\right)^2}{4\,E\,I} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This half value is due to the author's M_r uniformly decreasing from unity to 0. If M_r were displaced by the moment unity, the true value of angular rotation for the free end of the cantilever would result at once.

Again, the author's method may be used to determine the work expended in bending a beam subjected to loading at right angles to its axis. The stress, s, due to bending only, is given in the second term of the second member of the third unnumbered equation preceding Equation (4) of the paper. That value substituted in the author's Equation (3), after writing da for A and dl for l, will give Equation (5):

$$\alpha = \sum \int \frac{r\,M_s\,y\,d\,l}{E\,I} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The value of r is:

$$r = \frac{M_r\,y}{I}\,d\,a \qquad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Equation (5) may then be written:

$$\alpha = \sum \int \frac{M_s\,M_r\,(y^2\,d\,a)\,d\,l}{E\,I^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Mr. Burr.

The expression, $\int y^2 \, d\, a$, is, of course, the moment of inertia, I, of the cross-section of the beam. Hence, Equation (7), and also Equation (1), may be written:

$$d\, \alpha = \frac{M_s \, M_r \, d\, l}{E\, I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The work performed throughout the entire beam will then be shown by Equation (9):

$$\text{Work performed} = \int \frac{M_s \, d\, \alpha}{2} = \frac{1}{2\, E\, I} \int M_s^2 \, M_r \, d\, l \ldots\ldots (9)$$

The correct usual expression for the work performed in bending a beam, that is, the resilience, is as shown in Equation (10):

$$\text{Work performed} = \frac{1}{2\, E\, I} \int M_s^2 \, d\, l \ldots\ldots\ldots\ldots (10)$$

Equation (9), therefore, is correct only when M_r is unity.

Although the preceding criticism relates only to members or beams subjected to flexure, obviously the use of the author's Equation (3) is limited in its applications to those cases only in which the stress, r, is produced by the application of a couple with a unit moment to the sections covered by the summation.

It may be justly urged that since the moment, M_r, must be considered unity, its use as an analytic device is superfluous and unnecessary. So far as members subject to flexure only are concerned, nothing could be simpler or more direct in application than the old and standard formula given by the writer's Equation (2). This formula has been used, for many years, for angular displacement or rotation of the longitudinal axis of a bent member, especially in connection with the treatment of arched ribs.

Just what is meant specifically by the "new principle" is not obviously clear. The use of analytic expressions for the work performed in stressing both beams and pieces subject to direct stresses of tension or compression in the determination of deflections and for other similar purposes, is quite old. It would also seem to be much more simple to write the author's Equation (4) in correct form, by a direct equation between the external work performed and the sum of the two expressions for work, one covering members subject to direct tension or compression, and the other covering those subject to bending. This would render superfluous and eliminate the analytical work preceding Equation (4).

In passing, the surprising statement of the author, following his Equation (14), may be noticed. In referring to that equation, he writes, "which is the most general form of the Theorem of Three Moments." The most general form of the "Theorem of Three Moments", as is well known, provides for a variable moment inertia, a curved or broken center line of beam or truss, and other features

which are not even recognized in Equation (14). In fact, that equation is really a highly restricted form of equation expressing the "Theorem of Three Moments." Mr. Burr.

A. C. Janni,* M. Am. Soc. C. E. (by letter).—It has been already remarked by others that the conception of the angular displacement, or rotation, of a cross-section of a solid body, as well as the derivation of the relative formula, is not new; indeed, it is as old as the conception of the linear displacement of a point of a solid body, together with its relative formula. The claim to novelty in this case, therefore, seems to be quite out of place. Any good textbook will give some method of deriving this formula from the general expression of the "virtual work", either by the theorem on the "derivative of work", or by invoking the principle of "least work." Mr. Janni.

Of course, in using this formula, any book of higher engineering does not need to explain fully its origin, for it is assumed that whoever consults such books, is already familiar with the theory by which this formula has been worked out. That this is the logical assumption of the authors of such works is noted, for instance, in Grimm's book on Secondary Stresses, which is one of the only two mentioned by Professor Swain in his paper, as having been consulted by him on this matter, and which would justify his belief that the principle treated by him is new. In Chapter VII of this book, the author, speaking about the principle of "virtual work", states "with which the reader is supposed to be familiar."

That Professor Swain has been able to work out from this established formula a few plain cases of equations of equilibrium is not to be wondered at, for the derivation of the formula itself was worked out long ago from these equations, as any student in any engineering school should know.

After these considerations, there does not seem to be any necessity or use for the Profession to go into details and analyze the interpretation as given by Professor Swain to an already established principle and formula, for, at the present state of the science, this interpretation, at its best, would hardly be used by a modern engineer, for it is obsolete, as will be shown later by comparison.

Although the solution of this problem, considered by itself, has a doubtful practical value in most cases of equilibrium, still it will be demonstrated that by the theory of the "ellipse of elasticity", it is possible to obtain the desired solution in a manner which leaves no doubt as to the superiority of modern methods over the old ones. A few remarks will precede the exposition of the method.

To any elastic body there corresponds a certain quantity, or coefficient, called "elastic weight", which is a function of its length, its cross-section, its moment of inertia, and of the modulus, E, of the

* New York City.

Mr. Janni.

material.* It will be shown, in each case, how this elastic weight is to be computed.

The "elastic center" is the center of gravity of the elastic weights of a given body. If the elastic weight of a certain elastic system is concentrated, or applied, at the elastic center of the body, and a force, P (Fig. 33), is acting on the free end of this system, the following mathematical truth can be demonstrated:

The angular rotation α, of the free end of the elastic body, with respect to its fixed end, is given by the force times the static moment of the elastic weight with respect to the line of action of the acting force. That is, if P (Fig. 33) is the force applied, G, the elastic center and its value as well, and d, the distance between the line of action of P and the point G at which the value, G, is applied, then:

$$\alpha = P \times G \times d.$$

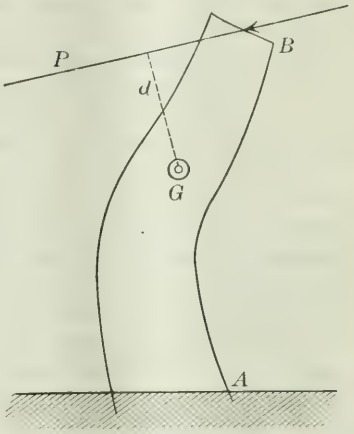

FIG. 33.

Consistent with brevity, a few cases will be shown, which will represent fairly the solution for almost all possible cases in practice.

Attention is called to the brevity, uniformity, and directness of this method, as well as to the very elementary kind of operations involved, in order to obtain the result.

Case 1.—Let AB (Fig. 34) be the geometrical axis of a beam, fixed at one end, A, and free at B, and let P be a vertical force applied at B. The inclination, α, of the tangent at B' is desired.

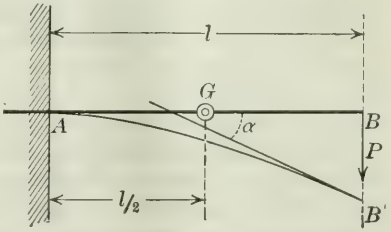

FIG. 34.

* The geometrical axis of a solid, usually, is not a straight line, and its cross-section is not constant. In this case, the solid is to be divided into parts, and the elastic weight of each part, or element, is to be computed separately, as will be shown later.

These elementary elastic weights are designated, usually, by the letter, w, with a numerical progressive index, and each is supposed to be applied at the center of gravity of its own element.

The elastic weight of the whole body is the summation of these elementary elastic weights.

Considering these w's as forces applied to the various centers of gravity, the position of the center of gravity of the entire system of forces, w's, may be determined.

This point is called the "elastic center", and the whole elastic weight is supposed to be applied at this center.

The position of this point, usually, is indicated by the letter, G, as is its value, so we have:

$$w_1 + w_2 + w_3 + \ldots + w_n = \Sigma_1^n w = G.$$

Mr. Janni.

Assuming that:

E = modulus of elasticity,

I = moment of inertia = constant, and,

A = area of cross-section = constant,

then the elastic weight of the whole beam is given by $G = \frac{l}{E\,I}$, and if it is supposed to be applied at G (the center of gravity of the elastic system), by the principle previously stated, we obtain:

$$\alpha = P \times \left(\frac{l}{E\,I} \times \frac{l}{2} \right) = \frac{P\,l^2}{2\,E\,I}$$

which result solves the problem.

Let us suppose now that the cross-section, A, is not constant, as is shown in Fig. 35; then, in order to take this variation into account, the geometrical axis, AB, will be divided into a number of parts, such that each of them may be considered, without sensible error, as a prism.

Then, find the $\frac{\Delta\,l}{E\,I} = w$ for each of these parts and, assuming that each w is a vertical force applied correspondingly at 1, 2, 3, ..., determine the position of their center of gravity (elastic center), G.

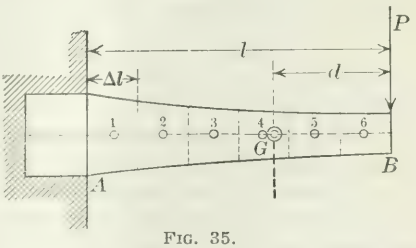

FIG. 35.

Now, remembering that the total elastic weight, G, is to be considered as applied at G, and remembering, also, the previously mentioned principle, we may write:

$$\alpha = P \times G \times d$$

where:

$G = w_1 + w_2 + \ldots + w_6 = \Sigma^6_1\, w$

Case 2.—Angular Rotation of the Free End of a Three-Member Frame.—Let Fig. 36 represent a three-member frame, comprising a girder supported at its ends, A and B, by posts, CA and DB, the girder connections at A and B being rigid, the post, CA, rigidly fixed at C, and the post, DB, on rollers at D.

Let us assume, as is usual, that the system has no dead load, and let us remove the ground support at D. Assuming, also, that E is the modulus of elasticity, and I is con-

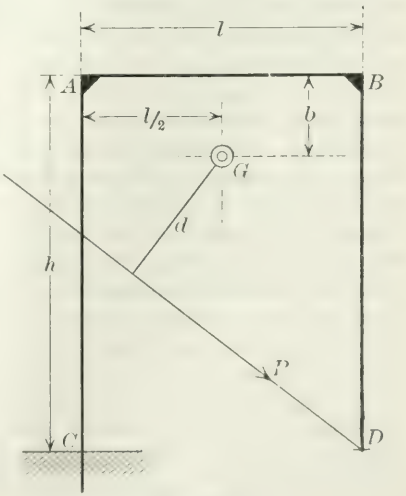

FIG. 36.

Mr. Jaunl

stant for all three members, then the elastic weight for each post is given by $\frac{h}{E\,I}$, and that of the girder by $\frac{l}{E\,I}$; therefore, the total elastic weight of the system will be:

$$G = \frac{l}{E\,I} + 2\,\frac{h}{E\,I} = \frac{1}{E\,I}\,(l + 2\,h)$$

The elastic center, G, of the three elastic weights, each of which being applied at the middle point of the corresponding member (see the first problem of Case 1), for the cross-section, A, is supposed to be constant, will lie on the vertical axis of the symmetry of the frame, and at a distance, b, from AB, given by:

$$b = \frac{2\,\frac{h}{E\,I} \times \frac{h}{2}}{\frac{1}{E\,I}\,(l + 2\,h)} = \frac{h^2}{l + 2\,h}$$

A force, P, applied at D, will cause a rotation of the section, D, which, by remembering the previously stated principle, is given by:

$$\alpha = P \times \left[\frac{1}{E\,I}\,(l + 2\,h) \times d\right] = \frac{P \cdot d}{E\,I}\,(l + 2\,h)$$

which is the result sought.

At this point, it seems to the writer that some remarks may be made, which are rather interesting.

Suppose that the force, P, revolves about D, then the angle, α, will vary with the distance, d. If, therefore, the line of action of P passes through G, then d is zero and, consequently, $\alpha = 0$. In this case, then, the section, D, does not perform any angular rotation.

In the previously mentioned theory, it is demonstrated that, in this case, Section D performs only a linear displacement, in a certain direction. The amount of this linear displacement and its direction can be determined, but it is not thought necessary to enter this field any further.

Let us suppose that the force, P (Fig. 37) is not applied at the end D, but at some other point of the frame, say, at the middle point, N, of AB. Then this force, as far

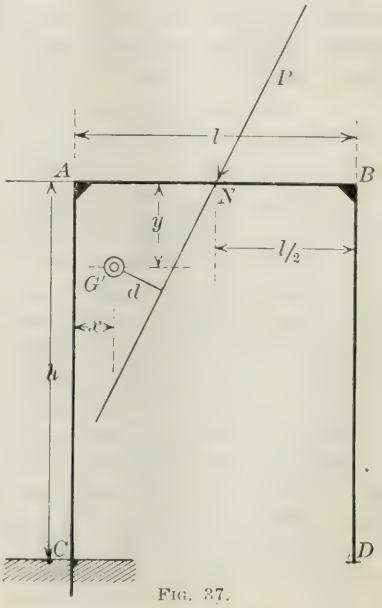

Fig. 37.

as internal stresses in the frame are concerned, has no effect on the half frame at the right of force, P, that is, on NBD. Mr. Janni.

In this case, we determine the following elastic weights:

$$\text{Elastic weight of } A\ C = \frac{h}{E\ I}$$

$$\text{“ “ “ } A\ N = \frac{l}{2\ E\ I}$$

$$\text{Total elastic weight} = \frac{1}{E\ I}\left(h + \frac{l}{2}\right).$$

The position of the center, G', is, evidently, given by:

$$x = \frac{\frac{l}{2\ E\ I} \times \frac{l}{2}}{\frac{h}{E\ I} + \frac{l}{2\ E\ I}} = \frac{l^2}{2\ (2\ h + l)}; \quad y = \frac{\frac{h}{E\ I} \times \frac{h}{2}}{\frac{h}{E\ I} + \frac{l}{2\ E\ I}} = \frac{h^2}{2\ h + l}.$$

Therefore:

$$\alpha = P \times \left[\frac{1}{E\ I}\left(h + \frac{l}{2}\right) \times d\right] = \frac{P\ d}{E\ I}\left(h + \frac{l}{2}\right).$$

Here, also, if the line of action of P passes through G', the section, N, and, consequently, the end, D, of the frame does not perform any rotation, but only a linear displacement.

Incidentally, by inspecting Figs. 36 and 37, it may be seen that it is always possible to regulate the force, P, in each case, as to its intensity and line of action, so that the rotation, α, of Section D is the same in each case.

It can be seen also that by the use of this principle in the solution of problems of this kind, apart from the fact of the very elementary operations required, one gets also an absolute and easy control of the behavior of the elastic system.

Case 3.—Let AB (Fig. 38), represent an arched solid, the law of generation of which may be any, and let us suppose that it is fixed at B and free at A (the terminal section of the solid). The angular rotation of the end section, A, is required, in the hypothesis that a force, P, acts on the whole solid at A (this force may be imagined as acting on A, away from A, but rigidly connected with it). The solid is divided into any number of parts (seven in this case), and the corresponding seven elastic weights are computed.

Imagine that these elastic weights, $w_1 = \frac{\Delta\ l}{E\ I_1}, w_2 = \frac{\Delta\ l}{E\ I_2}, \ldots w_7 = \frac{\Delta\ l}{E\ I_7}$, are forces, each applied at the corresponding center of gravity 1, 2, 3, . . . 7 of the voussoir or element, and determine the position, G, of the elastic center (center of gravity of the elastic weights).

Mr. Janni.

Then, if P is the acting force, the angular rotation, α, of A, by making $G = \Sigma_1^7 w = w_1 + w_2 + \ldots + w_7$ is given by:

$$\alpha = P \times (G \times d) = P \times G \times d,$$

which value solves the problem.

If P' is the force applied to the elastic body then, as in Case 2, the terminal section, A, will not perform any angular rotation, but a linear displacement, the amount and direction of which can be determined, as previously stated.

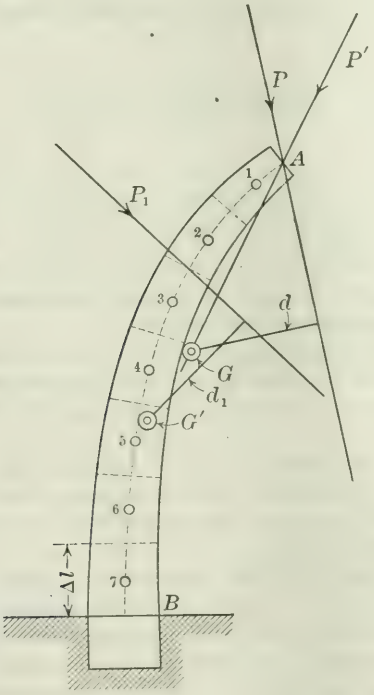

FIG. 38.

In the case in which the force applied is P_1, then, similarly, as in Case 2, the position of G' must be determined, as well as its value; remembering, however, that, in this case, the elastic weights to be omitted are $w_1 = \frac{\Delta l}{E I_1}$ and $w_2 = \frac{\Delta l}{E I_2}$, for the force, P_1, has no direct effect on the two voussoirs, 1 and 2.

The position, G', the center of gravity of the system 3, 4, 5, 6, and 7, being determined, and the value of d_1 found, we will have: Mr. Janni

$$\alpha = P_1 \times (G' \times d_1) = P_1 \times G' \times d_1$$

which solves the problem.

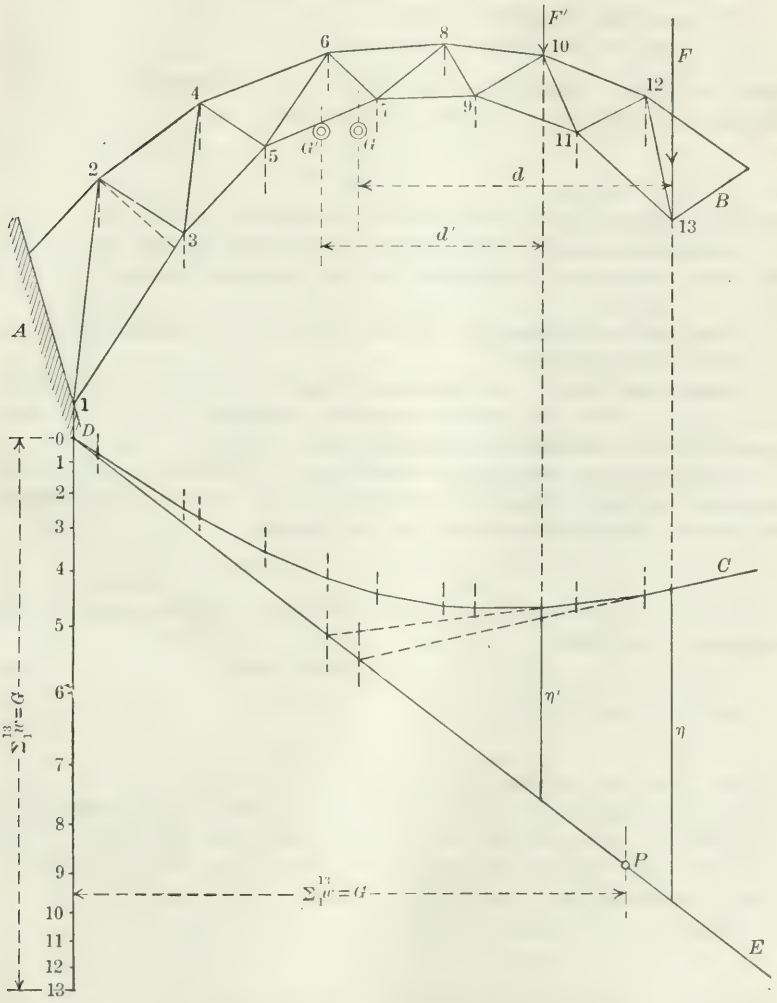

FIG. 39.

Case 4.—Finally, let AB (Fig. 39) be a latticed arch, having the end A fixed and the end B free. Let us find the angular rotation of B when a force acts on this elastic system.

Mr. Janni. In the solution of this problem, the most general method will be used.

If we put:

l = length of 1–3,
r = distance between Point 2 and Line 1–3,
E = modulus of elasticity, and,
A = cross-section area of Member 1–3,

then, the elastic weight corresponding to the Member 1–3, by the theory of the ellipse of elasticity, is given by:

$$w^2 = \frac{l}{E\ A\ r^2}$$

which is supposed to be applied at its corresponding pole, 2.

In the same way, the other elastic weights of the system, w_1, w_3, ... w_{13}, may be computed and applied to the corresponding poles, 1, 3, ... 13.

Now, draw a vertical line and lay off on it, at a convenient scale, all these elastic weights.

Choose any pole, P, at a distance from this vertical line equal to $\Sigma_1^{13} w = G$, and on the verticals passing through 1, 2, ... 13, construct the corresponding funicular polygon, CDE.

Let us suppose now that a vertical force, F, is acting at B on the whole system, then, as has been previously stated, the rotation performed by Section B is to be computed by the formula:

$$\alpha = F \times (G \times d) = F \times G \times d$$

If η is the intercept on the vertical line of F, determined by the polygon, CDE, then:

$$G \times d = \eta \times (\Sigma_1^{13} w = G)$$

therefore, we may write:

$$\alpha = F \times G \times \eta \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

which solves the problem.

Let us suppose, now, that the force F' is acting on this system, then omitting the elastic weights, w_{11}, w_{12}, w_{13}, on which the force, F', has no direct bearing, the new system will have a new elastic center, G', and its value will be $G' = \Sigma_1^{10} w$.

Identically, as before, we will have:

$$\alpha = F' \times (G' \times d') = F' \times G' \times d'$$

and, if η' is the intercept, as shown in Fig. 39, then:

$$G' \times d' = \eta' \times (\Sigma_1^{13} w = G) = \eta' \times G$$

therefore:

$$\alpha = F' \times G \times \eta' \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Mr. Janni.

By inspecting Equations (1) and (2), it may be seen that there is no necessity, in any case, to determine the position of the elastic center, but that the construction of the polygon, *CDE*, is sufficient for the solution of the problem, no matter what the point is on which the external force is supposed to act. That is, we may write the expression, α, in its most general form, thus:

α = Acting force $\times$ Total elastic weight $\times$ Intercept on the vertical of the acting force by the funicular polygon, *CDE*.

This kind of a problem could hardly be more elegantly and simply solved than by the use of this modern theory.

Mr. Eddy.

H. T. Eddy,* Esq. (by letter).—There is a certain limitation to Mr. Maney's method which seems to make his arch analysis inapplicable to the transverse frames of ships; and even were this obstacle removed in some way, the moment of inertia of reinforced concrete arch sections suffers such discontinuous changes under stresses, due to combined bending and compression, depending on which of these two kinds of stress predominates, that the method would require radical emendation to make it afford a suitable basis for investigating the design of concrete ships.

The limitation just mentioned may be expressed by stating that the method is necessarily restricted to cases where every one of the applied forces of the arch loading acts in a direction perpendicular to the line, *AB*, joining the ends, *A* and *B*, of the member which acts as an arch. That this is necessarily so is evident from the fact that the moment, *M'*, of all the applied forces, including the reactions of the supports, but excluding the restraining moments at the supports, must be calculated, as is correctly stated by Mr. Maney, on a statically determinate basis. This requires that the magnitudes of both the opposing thrusts, F_A and F_B, acting along *A B*, at *A* and *B*, respectively, shall be known.

These thrusts, however, are statically indeterminate in case there is a component, *P*, of any force which is applied to the arch rib at any point that acts in a direction parallel to *AB*, since, in that case, the algebraic sum of the reactions along *AB* that hold *P* in equilibrium is $F_A + F_B = P$, in which the actual magnitudes of F_A and F_B are statically indeterminate, not only for an arch, but for a straight member as well. For it is evident that a force, *P*, applied longitudinally to a straight piece, *AB*, at some point, *Q*, intermediate between its extremities taken as fixed, will awaken reactions at *A* and *B* which are not statically determinate, but are dependent on the relative length and elasticity of the segments, *QA* and *QB*. This is also true of an elastic arch, *AQB*.

* Minneapolis, Minn.

Mr. Eddy.

In general, the thrusts, F_A and F_B, at the supports are statically determinate only in case $F_A + F_B = 0$, that is, in case no one of the applied forces has a component parallel to AB.

However, since hydraulic pressures act normal to the exterior surface of a ship, any curved member of a transverse frame functioning as an arch rib and subjected to these pressures must be subjected to components of those pressures, which act along the line, AB, joining its extremities. Hence, the proposed method based on zero pressures parallel to AB is inapplicable to curved members of ship frames.

Just how this limitation restricts the proposed analysis may be made more clear by the following discussion of the principles involved in establishing and interpreting Mr. Maney's Equation (1), which is an expression in algebraic form of the fundamental principles of the loaded elastic arch with level supports, as first made public by the writer in his paper on "Certain New Constructions in Graphical Statics."*

In considering arch action in general for vertical loads, it may be stated that when the axis of the arch rib coincides in position and direction with the line of resultant pressures due to all the applied forces—which line is identical with the equilibrium polygon of those forces—then the structure constitutes an equilibrated arch, as it is called by Rankine, and the arch action of a rib of this shape provides, by reason of its shape alone, all the resisting moment that is required in order to hold in equilibrium the moments of the applied forces, and no bending moment whatever is exerted by the arch rib itself in resisting the applied moments. The entire stress in the rib in this case consists of direct compression along its axis. An equilibrated arch coincides with a catenary curve of the loads placed in an upright position.

Let A and B be the points of support on the axis of an arch rib, AQB, as in Fig. 29, in which AB is horizontal and Q any point on the curved axis. If the point, Q, does not also lie on the equilibrium polygon of the applied forces corresponding to the actual thrust, F, of the arch, then this is not an equilibrated arch, because the curved axis of the arch deviates at this point, Q, from the equilibrium polygon and, consequently, the arch rib itself must sustain some bending moment, M, and thus furnish part of the moment that resists the total moment, M', due to the applied forces.

* Read before the American Association for the Advancement of Science at Buffalo, N. Y., in 1876, and, later, published in the *American Journal of Mathematics*, Vol. I, No. 1, and in *Van Nostrand's Engineering Magazine*, and republished in the writer's book entitled "Researches in Graphical Statics", New York, 1878.

Let Q' be the point where the vertical at Q cuts the equilibrium polygon, and Q'' the point where it cuts AB; and let r = the total rise of the arch, and $QQ'' = vr$, then: Mr. Eddy.

$M' = F \times Q'\ Q'' =$ the total moment at Q of all the applied forces as found for a simple beam, AB, that is, the moment due to all the forces and reactions except end moments.

$M = F \times Q'\ Q =$ that part of the moment M' resisted by the bending moment in the arch rib.

$Fvr = F \times Q\ Q'' =$ that part of M' resisted by arch action by virtue of the thrust, F, along AB, and the equal horizontal component of axial compression at Q.

$Fvr + M =$ total resisting moment due to arch action and to the bending moment in the rib.

Let $AB = L$, $AQ'' = aL$, and $BQ'' = bL$, and let M_A and M_B be the restraining moments at A and B, respectively. Then, aM_B and bM_A are the resisting moments at Q due, respectively, to M_B and M_A.

When the applied forces are perpendicular to AB, the conditions of equilibrium of the arch rib, AQB, evidently require at every point, Q, that the sum obtained by adding the bending moment, M, of the rib itself to the moment, Fvr, representing the arch action, shall be equal to the total statical moment at that point due to all the forces applied to the arch, including not only the vertical loads and the reactions of the supports, but also the effects of the restraining moments at A and B.

Hence, we obtain Mr. Maney's Equation (1), namely,

$$M + Fvr = M' + aM_B + bM_A$$

in which the intrinsic sign of F has been reversed for convenience.

The total statical moment, M', may be taken to consist of two parts, as follows: One part due to the loads and the vertical components of the reactions which may be constructed by drawing the equilibrium polygon for a simple beam supported at the ends with a pole distance = F, and carrying the applied loads. The vertical ordinate at any point, Q, of this polygon multiplied by F will give the moment at Q of the loads and the vertical components of the reactions at A and B. The other part of M' is Fvr, the moment of the horizontal component of either reaction. Since, however, both sides of Equation (1) include the same quantity, Fvr, the first side explicitly, and the other side implicitly in M, they might be expressed so as to cancel each other, and it is, consequently, wholly immaterial what value may be assigned to F, the horizontal component of the

Mr. Eddy.

reaction at either support, or, what is the same thing, the direction of one reaction may be arbitrarily assumed provided all the loads act perpendicularly to AB, for Equation (1), in fact, has to do only with the moment of the vertical forces.

As already stated, another question of fundamental importance in reinforced concrete under combined compression and bending is a correct method of calculating the value of the moment of inertia of any composite cross-section, especially at points where the applied moments are large.

The value of I in a combination of concrete and metal depends, not on the area of the concrete and steel alone, but on the condition of stress normal to the section. If the condition of stress is that of simple flexure, with no longitudinal thrust, then I depends entirely on the steel and its position in the matrix, together with that part of the concrete which is under compression. If, however, the entire section is under compression, then I depends on the total area of the concrete and the area of the steel combined.

As an example, consider the moment of inertia of a column supporting a floor. If the load on the column is such that the total area of the concrete is under compression, then this total area enters into the moment of inertia of the column section and must be taken into consideration in the apportionment of moments between the column and the floor. Whereas, if the load is insufficient to cause the column to be under compression throughout its full area, that part of the section of the concrete which is under tension, does not enter into the determination of the moment of inertia, but must be disregarded entirely. Deflections may vary in such constructions nearly 100% with the variations of the moment of inertia of the same cross-section of metal and concrete depending entirely on the state of longitudinal stress in the columns. Under Mr. Maney's discussion, no attention is paid to this important matter, and the validity of his analysis must depend on the correctness of the determination of I at the different sections considered, even in cases where the applied loads have no components parallel to AB.

Professor Burr's objections and emendations do not seem to be well taken in his attempt to identify Dr. Swain's formula for the actual rotation, α, which occurs at any section of a piece, due to the loads and restraints to which the whole piece is subjected, with the total change of slope or curvature between two given sections of the piece in accordance with Rankine's formula:

$$i_2 - i_1 = \int_1^2 \frac{M_s \, d\, l}{E\, I}$$

where i_1 and i_2 are the slopes at given sections, A and Q, of a piece, AB.

Mr. Eddy.

It is only in certain particular cases that the change of slope, or the total curvature produced between A and Q, as found by this equation, is the same as the rotation at Q, since they are the same only when the piece is fixed at A in such a manner that no rotation occurs at that point. Then, and then only, do the formulas for slope and rotation give identical results.

If, however, the piece is fixed at A and free at B, then a unit moment applied at Q must be held in equilibrium by a unit-resisting moment at A, and there is a unit-bending moment, M_r, produced by it at every point of AQ, and no bending moment, M_r, in BQ.

It is self-evident, however, that, in Dr. Swain's Equation (5), M_r cannot in general be equal to unity, as Professor Burr asserts, because the limits of the integrations in which it occurs extend, all told, from A to B, although the section where the rotation occurs is at Q, because the integration above for the change of slope extends from A to Q only. In other words, if $M_r = 1$ at all points of AB that would require:

$$\int_A^Q \frac{M_s\, d\, l}{E\, I} = \int_A^B \frac{M_s\, d\, l}{E\, I},$$

where the first member is the change of slope and the second member is the rotation, in case M_r is assumed to be equal to unity.

Hence,
$$\int_A^Q \frac{M_s\, d\, l}{E\, I} = \int_A^Q \frac{M_s\, d\, l}{E\, I} + \int_Q^B \frac{M_s\, d\, l}{E\, I},$$

$$\text{or, } 0 = \int^B \frac{M_s\, d\, l}{E\, I},$$

which is absurd.

The conclusive demonstrations of Castigliano and Müller-Breslau, as shown clearly by Professor Church and Mr. Mensch, in their extended and original elucidations, make it certain that Dr. Swain's formula contains no material error when properly interpreted.

It should be particularly noted that the value of M_r depends on the same restraints and the same manner of support that are effective to determine the distribution of the applied moments that produce the moment, M_s, actually operating.

For example, in a beam of length, $AB = L$, uniformly loaded and rigidly fixed in direction at both A and B, the moment polygon due to a unit couple applied at Q, when $AQ = aL$ and $BQ = bL$, consists of two horizontal lines such that in the segment, AQ, we have $M_r = b$ and, in BQ, we have $M'_r = -a$. The sides of the polygon are straight at the right and left of Q, because the only applied forces in the polygon, the ordinates of which represent M_r, are the forces of a couple of unity applied at Q. The fixity of the ends requires that the positive moment area shall equal the negative area, as they do in this case, namely, $aLM_r = -bLM'_r = abL$.

Mr. Cain.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—Professor Swain published in the *Journal* of the Franklin Institute, for February, March, and April, 1883, a paper "On the Application of the Principle of Virtual Velocities to the Determination of the Deflection and Stresses of Frames", the method being based principally upon the investigations of Mohr and Winkler. It is interesting to note that the author has taken up the subject again after 36 years, and has applied the method to finding the rotation of a section of a beam or truss, under given loading; eventually applying the results to deducing the "Theorem of Three Moments" and in making other interesting applications. The formulas and their applications are sound in theory and are neatly and concisely presented.

It is true, as pointed out by both Professor Church and Mr. Mensch, that the author's Equation (6) is implicitly contained in a theorem of Castigliano,† but the author deserves our thanks for putting it in this definite, explicit form, where no differentiation is called for. The method of the author does not require, as in the "moment area" method, that the beam or frame shall be fixed in direction anywhere, but it does require immovable abutments, and that all the reactions at the abutments, whether due to the loads alone, or to the fictitious couple, shall be included in the forces acting on the structure, which is then taken as a free body, in equilibrium under the loads and reactions, including any restraining couples acting at the abutments.

When these conditions are observed, the author's Equation (6) gives the rotation of a section of a beam or truss when the summation (or integration) covers the whole beam or all the members of the truss. Thus, for the simple beam, Fig. 2, subjected to a load P and a couple, the moment of which is $M = 1$, applied at the right end, the beam is free under these loads with their corresponding reactions. Then, when the integration covers the entire span, Equation (5) gives the rotation (in radians) of a vertical section where the couple is applied. It would be erroneous to apply the method to only a part of the truss, for reasons given later.

By Castigliano's method, where an actual force at the point, acting in the required direction, (or an actual couple at the desired section), is not available, an arbitrary force (or couple) is applied at the point (or section), which is eventually taken equal to zero; then, the partial derivative of the internal work of the structure with respect to the force gives the deflection, at that point, in the direction of the force, and the partial derivative of the internal work with respect to the

* Chapel Hill, N. C.

† The writer first had the pleasure of introducing Castigliano's masterly treatise on "Systèmes Elastiques" to the notice of American engineers in a paper, published in *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 265, April, 1891. In this paper, some original demonstrations of the method of deflection and least work were given, besides a general résumé of Castigliano's fundamental theories and methods, with some applications.

moment of the couple, supposed applied at a section, will give the rotation of that section. Mr. Cain.

Thus, in the case of arches subjected to vertical or inclined loads, there are introduced at a point on the axis a horizontal force, w, a vertical force, w', and a couple of moment, m; then, if L is the internal work of the arch, expressed in terms of w, w', and m and the given loads, then, taking partial derivatives and, in the end, making w, w', and m equal to zero,

$$\frac{dL}{dw} = \text{horizontal deflection of point;}$$

$$\frac{dL}{dw'} = \text{vertical deflection of point;}$$

$$\frac{dL}{dm} = \text{rotation of section at point,}$$

all for the actual loads only.

In the case of a long column subjected to an inclined load at the top, eccentrically applied, the general solution is effected with the use of only the two loads w and w'. In other cases, only one load or one couple may be used. To give an illustration of Castigliano's method, take the case of the simple beam, shown in Fig. 3, where a load, P, is applied at a distance, a, from the left abutment and an arbitrary couple, which is taken left-handed, is supposed applied to the beam at the load, say, by means of a bar bolted to the beam. If M_1 is the moment of the couple applied at P, its left reaction acts up and equals $\frac{M_1}{l}$, and its right reaction acts down and also equals $\frac{M_1}{l}$.

Call M the moment of all forces from the left abutment to a section x ft. from the left abutment, or x_1 ft. from the right abutment.

$$o < x < a,\ M = P\frac{l-a}{l}x + M_1\frac{x}{l};\ \therefore \frac{dM}{dM_1} = \frac{x}{l}$$

$$o < x_1 < (l-a),\ M = P\frac{a}{l}x_1 - M_1\frac{x_1}{l};\ \therefore \frac{dM}{dM_1} = -\frac{x_1}{l}.$$

Now, neglecting shear, the internal work for the beam is given by the well known formula

$$L = \frac{1}{2}\int_0^l \frac{M^2}{EI}\,dx$$

whence the rotation of a vertical section at the load is

$$\alpha_1 = \frac{dL}{dM_1} = \int_0^l \frac{M}{EI}\frac{dM}{dM_1}\,dx.$$

On putting $M_1 = 0$ in the values of M given above and then substituting the values of M and $\frac{dM}{dM_1}$ pertaining to the parts of the beam to

Mr. Cain. the left and right of the load and integrating between the limits $x = 0$ and $x = a$ for the left part, and between $x_1 = 0$ and $x_1 = (l - a)$ for the right part, we are conducted to the integral given by the author, and thus to his value

$$\alpha_1 = \frac{P\,a\,(l - a)\,(2\,a - l)}{3\,E\,I\,l}.$$

As this is negative, the rotation of a vertical section at the load is in the opposite direction to that of the couple, or clockwise. Since the influence of shear on bending was neglected, this value of α_1 likewise gives the slope of the elastic curve (in radians) at the load, which is thus downward to the right.

It is seen that the values of $\frac{d\,M}{d\,M_1}$ above are the same for any value of M_1 and therefore for $M_1 = 1$, in which case the moments due to this couple of moment 1 are exactly the values given by the author, so that $\frac{d\,M}{d\,M_1} = M_r$ as defined by the author. In fact, this is generally true for all cases, since M is always a linear function of M_1 for any structure, hence the author's Equation (5) is identical with Castigliano's formula

$$\alpha = \int \frac{M}{E\,I}\,\frac{d\,M}{d\,M_1}\,dx$$

for any beam, straight or curved.

As to the more general formulas for Δ and α, Professor Church, in his discussion, has so thoroughly pointed out the equivalence of the author's and Castigliano's formulas, that it seems superfluous to make any further comment thereon.

Where an external couple acts at the required section, it is not necessary to add the arbitrary couple. Thus, for the beam supported at the ends and subjected only to a couple, M_2, at the right end, Fig. 6, we have

$$M = \frac{M_2}{l}\,x; \quad \therefore \frac{d\,M}{d\,M_2} = \frac{x}{l}$$

and the rotation of a vertical section at the right end is

$$\alpha = \int_0^l \frac{M}{E\,I}\,\frac{d\,M}{d\,M_2}\,dx = \frac{1}{E\,I}\int_0^l \frac{M_2\,x^2}{l^2} = \frac{M_2\,l}{3\,E\,I}.$$

Since α is positive, the rotation is in the direction of that of the couple, or counter-clockwise. The same final result would be obtained by the use of an arbitrary couple at the right end, but the work is slightly greater.

Mr. Mensch, in his comprehensive discussion, premises that the theory, in its most general form, is based upon the law that Mr. Cain.

$$\text{External work} = \text{Internal work};$$

where "external work" includes that of the loads and reactions (including restraining couples, if any). It is thus evident that the usual formulas are only true for immovable abutments, where the reactions do no work.

It may be well to add that for a simple beam with vertical loads, if the left abutment sinks e ft., its vertical reaction, R, does the work $-R\,e$. The total sinking of any load, P, on the beam with reference to the ground, is made up of two parts, a part, Δ_1, due to the rotation of the straight beam about the right end, from the sinking of the left end e ft.; and a second part, Δ_2, due to the elastic deflection, referred to immovable abutments. Thus, the "external work" in this case is

$$-R\,e + \Sigma\,P\,(\Delta_1 + \Delta_2)$$

where the sum is to be taken for all the loads on the beam. Similarly, the case where both abutments sink can be dealt with. If both abutments sink an equal amount, then the external work is simply equal to $\Sigma\,(P\,\Delta_2)$.

If there is a restraining left-handed couple, the moment of which is M_1, at the left end of a loaded beam and the vertical section there turns α radians to the right, then it is easily shown that the work of the couple is $-M_1\,\alpha$. For gradually applied loads, the reaction and restraining couple will be gradually applied, so that the "exterior work" will be one-half of that given above, or

$$\frac{1}{2}\left[-R\,e - M_1\,\alpha + \Sigma\,P\,(\Delta_1 + \Delta_2)\right]$$

which must be equated to the "internal work"; or, the convenient derived formulas of Mr. Mensch may be applied.

Simply to illustrate the application of the general method, consider again the simple beam shown by Fig. 3, which carries a single load P, distant a ft. from the left abutment. Then, for immovable abutments, if Δ is the deflection at the load for a gradually applied load, the exterior work is $\frac{1}{2}\,P\,\Delta$, which equals the interior work

$$\frac{1}{2}\int_0^l \frac{M^2}{E\,I}\,dx$$

$$\therefore\ \Delta = \frac{1}{P}\int_0^l \frac{M^2}{E\,I}\,dx,$$

a well known formula.

Mr. Cain.

Here, to the left of the load,

$$M = P\,\frac{l-a}{l}\,x;$$

to the right of the load,

$$M = P\,\frac{a}{l}\,x_1.$$

Therefore,

$$\Delta = \frac{P}{E\,I}\left[\int_0^a \frac{(l-a)^2}{l^2}\,x^2\,dx + \int_0^{l-a} \frac{a^2}{l^2}\,x_1^{\,2}\,dx_1\right]$$

$$\therefore\ \Delta = \frac{P\,a^2\,(l-a)^2}{3\,E\,I\,l}.$$

To find α, the rotation of a vertical section at the load, pass a section through the beam just to the left of P and supply forces at the section equivalent to the action of the part of the beam to the right of the section on the part to the left, *viz.*, a left-handed couple the moment of which is $P\,\frac{a\,(l-a)}{l}$, and a downward acting force, $P\,\frac{l-a}{l}$, Fig. 40.

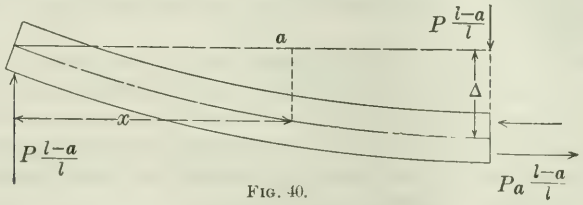

FIG. 40.

This latter force, with the left reaction, forms a couple of moment equal to that at the cut section; hence the system is one in equilibrium, and the exterior work equals the internal work, or

$$P\,\frac{l-a}{l}\,\Delta + \left(P\,a\,\frac{l-a}{l}\right)\alpha = \int_0^a \frac{M^2}{E\,I}\,dx.$$

Substituting the value of Δ, derived previously, writing

$$M = P\,\frac{l-a}{l}\,x$$

and, after effecting the integration, dividing through by $P\,a\,\frac{l-a}{l}$ and solving for α, there is obtained

$$\alpha = \frac{P\,a\,(l-a)\,(2\,a-l)}{3\,E\,I\,l}$$

exactly as given by the author.

Since $2a < l$, α is negative; hence the vertical section under the load rotates α radians in an opposite direction to that of the couple there, or to the right. Mr. Cain.

This, of course, is not the shortest solution of this problem, but it is given to illustrate principles and incidentally to show that the ordinary formulas for beams with immovable abutments do not apply to the segment of the beams, with its loads and reactions, as given in Fig. 40. They do apply, however, to one-half of the beam, when the load is at the center, to find the rotation of a vertical section at the abutment; for the half-beam is in the condition of a cantilever fixed at one end, with a horizontal tangent to the elastic curve, and loaded with $\frac{P}{2}$ at the abutment, acting upward, so that the solution, as in the case of the author's Case I, is quickly effected.

The writer, in conclusion, desires to express his great pleasure in reading the author's stimulating paper, which presents the subject so concisely and, at the same time, so clearly.

GEORGE F. SWAIN,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).— Discussion on this paper would naturally be directed to (1) criticism or commendation of the principle or the writer's presentation of it; or (2), whether, as the writer supposed when he presented it, it was novel. Mr. Swain.

There has been little criticism, except in two instances, of the principle itself, or of its presentation, and the writer here expresses his thanks for the commendatory remarks which have been made by many who have discussed it. As to the principle itself, it is simple and fundamental, and stands alongside the principle of deflection first given by Maxwell, and first published in this country by the writer, as a wholly similar method of finding rotation. There is no question as to its correctness, though one contributor does not admit it.

It should be stated at once that the writer did not claim anywhere that the problems which could be solved by the aid of the principle could not be solved otherwise, in some cases as easily as by its aid, and in some cases more easily than by it. The object was merely to present it as a fundamental principle in the theory of structures which he believed had escaped recognition by American engineers even if, as he suggested, it had been published before, although he was not aware and still does not believe that it had, in its general form. The discussion affords good evidence that it had not been recognized or appreciated, even if it had been known.

Since the paper was published, the writer has found—partly from the references quoted in the discussions, and partly in other ways—that, as he had surmised, the principle had been known in Europe, but only as applied to beams, and not as applied to trusses or combinations.

* Cambridge, Mass.

Mr. Swain.

He does not believe, however, that the principle had been at all recognized or used in America, either for beams or trusses, although it is referred to in one American book, but not explained or illustrated even there. The book referred to is "Bridge Engineering", Vol. I, by J. A. L. Waddell, M. Am. Soc. C. E., where, on page 231, after referring to the formula for the deflection of a beam,

$$\delta = \int_0^l \frac{M\, m\, d\, x}{E\, I},$$

it is stated:

"If it should be desired to find the amount of rotation of a given point of a beam, a couple with a moment of unity can be placed at the said point, and Equation (5) applied by changing δ to α (the angle in radians through which the point turns), the values of m being those produced by the unit couple".

The writer has not found in this book, or any other, any reference to its application to trusses.

Bearing in mind that the writer's paper showed a method of finding the angular rotation of the axis of a piece at any point, due to given loads, whether the piece is a straight or curved beam, or a truss member, or both, it may be well to consider some of the references given by those who claim that the principle is not new.

Professor Parcel states that the principle is not new, but he says it is little understood. He gives several references. The writer does not find in Molitor's work, on page 18 or pages 28-29, or elsewhere, any statement of the principle. Hartmann, pages 9-11, finds rotation only for solid beams, and on pages 24-33 there are only illustrations for solid beams, with no hint of its being a general principle. Mehrtens, at the pages cited, or elsewhere, gives no statement of the principle at all, but treats an entirely different problem, namely, that of finding the angular rotation of a line passing through any two joints of a frame. If a line, L, is drawn through any two joints of a frame, the distance apart of which is m, and if it be desired to find the angular rotation of this line under given loads, this may easily be done by the use of deflections. If a unit load is applied at one of the joints, perpendicular to L, and if this load causes stresses, t, in the members, the deflection of the point in the direction of the unit load due to the actual (not unit) loads will be, by the deflection formula,

$$\sum \frac{t\, s\, l}{E\, A}.$$

If a unit load is applied at the other joint perpendicular to L, but in the other sense, causing stresses, t', the deflection of this point under the actual loads will be

$$\sum \frac{t'\, s\, l}{E\, A}.$$

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The sum of these two quantities divided by m, will be the angular rotation of the line; but this will be the same as

$$\sum \frac{t'' s l}{E A},$$

in which t'' is the stress due to a load $\frac{1}{m}$ at each joint, or a unit couple, with one force at each joint. This is easy, and follows, from the deflection formula, but it is not the writer's principle at all.

If Professor Parcel thinks Mehrtens has given the writer's principle, he has failed to grasp that principle. Müller-Breslau is referred to by Professor Parcel. The writer has only the 1907 edition of his "Graphische Statik", but in it finds no statement of his principle. On pages 14-17 the rotation of a line through any two joints, just referred to, is what is found; and on pages 52-57 are historical notes, containing no statement of the principle. Müller-Breslau is also referred to by Professor Church, who cites Equation (54*a*) on page 133. This equation, however, refers only to solid beams, straight or curved, and is in a paragraph of Abschnitt II, which bears the title "Biegungsfestigkeit gerader und einfach gekrümmter Stäbe". His Equation (54*a*) is obtained from Equations (37) and (38), which, as the figures show, apply to solid beams, not trusses. Also, his Equation (54*a*) is

$$t_m = \int \frac{N n d x}{E I}, \text{ etc.},$$

which clearly shows that it applies to a beam. The writer's equation contains $\Sigma \int$, which is very different.

The writer has found no evidence that Müller-Breslau, or any other European writer, had perceived the principle, except as applied to beams, or, if they had, that they have stated it. Of course, it is easy to see, now that the writer has pointed it out, that the principle is implicitly contained in the work equation, but many things are hidden until they are made clear.

Professor Turneaure states that the principle is not new to him. This is not surprising, since it is so simple and obvious; but if Professor Turneaure had perceived it, he evidently did not think it important enough to mention, and the writer cannot in the least agree with his statement that it is "quite fully set forth" in his book. Indeed, the writer does not believe that anybody, reading the paragraph in Professor Turneaure's excellent book to which reference is made, would get even an inkling of the principle. Let any reader test this. In the first place, the paragraph he quotes deals with stresses in redundant members, and in this case the question of angular rotation is not involved. The quoted paragraph refers to structures which include

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"members acting as beams". This must mean cases in which the members are acted upon by transverse loads. It cannot refer to secondary stresses, for these are treated in a later chapter which begins with the statement, "It has been assumed heretofore * * * that all members of a truss are free to turn at the joints." What, then, is the meaning of Professor Turneaure's reference to "a bending moment of unity in the redundant member"? At what point in the redundant member? Presumably at the point where the rotation is to be determined. Professor Turneaure, however, nowhere gives any formula for finding rotation analogous to that for finding deflection. The writer's principle deals with a moment unity at the point where the rotation is to be determined, and shows how to find that rotation, and Professor Turneaure nowhere does this. The writer is free to confess that he does not clearly understand Professor Turneaure's quoted paragraph, but he is sure that it does not contain, or even suggest, his principle. Again, the writer has not found anywhere in Professor Turneaure's volumes any statement or illustration of the principle, even for beams.

Replying to Mr. Godfrey, the writer's object was to present a scientific principle and to illustrate its application. Mr. Godfrey thinks that "not one engineer in fifty doing practical designing could follow this paper". That, if true, does not speak very well for engineers, and would suffice to explain the low estimation in which many are held. He also thinks "there is no reason on earth why it [the method] should be applied in a practical design." Well, there was a time, not so long ago, when few engineers "doing practical designing" thought that pins in a bridge should be computed at all, or that the eccentricity of connections was of any importance, and similarly for many other things.

Mr. Godfrey must certainly have perceived that where members or frames are rigidly connected, it is necessary to compute or find an expression for the angular rotation at the points of connection. Sometimes a truss is connected to a beam, sometimes pieces in flexure are connected to other pieces in flexure; and if the engineer is to be something more than a "rule-of-thumb" man, able to solve new problems intelligently, he should have at command the scientific principles which will lead to a solution. The principle given by the writer will be found useful by any structural engineer who has the perception and ability to use it, and those who have not will continue to use "rule-of-thumb" methods, and will either make their structures too light, or waste their clients' money. The principle explained has simplicity for its chief merit, being as easily applied as the usual deflection formula, which few engineers "doing practical designing" knew anything about thirty years ago.

The writer quite agrees with Mr. Godfrey as to the desirability of simplifying construction and also mathematical processes, and thinks

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he has done the latter in his paper, as witness his demonstration of the Theorem of Three Moments. It seems, therefore, rather ridiculous to find Mr. Godfrey implying that the writer's paper tended to complicate mathematical processes. The writer also entirely agrees with Mr. Godfrey as to the necessity of common sense in engineering, and the importance of discriminating between important and trivial theoretical matters. He has preached and practiced this doctrine for more than thirty years; but common sense is not guesswork, and one cannot distinguish between important and trivial theoretical matters unless he understands theory. Certainly a man who could not follow through this paper could not so discriminate, and if engineers "doing practical designing" would give more study to simple fundamental principles, of which the paper illustrates one, there would be fewer of the "flimsy and wobbly buildings" that Mr. Godfrey refers to. There are undoubtedly many crimes committed in the name of theory, yet probably not so many as in the name of common sense. The important things are balance, perception, experience, judgment, and a knowledge of fundamental principles.

Professor Church's discussion, like everything he writes, is illuminating, and the writer thanks him for his contribution. The difference between the writer's Equation (4) and Müller-Breslau's Equation (54*a*) has been already alluded to. Professor Church's Equation (33) is not quite a correct reproduction of the latter, for reasons already explained.

Two of the discussions, those of Professor Burr and Mr. Jamni, criticize the principle itself, as to its correctness or value. Professor Burr has evidently not grasped the principle, and his discussion shows a number of misapprehensions. The principle is not restricted, as he thinks, to "that class of structures in which the axes of members are subject to angular displacement due to stresses of flexure or bending", but it applies to any structure, whether there is flexure or not, as illustrated in Case IV. Professor Burr, however, restricts his discussion to cases of flexure. His statement that the unit moment must be applied to each section is not correct, as is clearly shown by Case II. The unit moment is only applied at the point where the rotation is desired. If the case is that of a beam fixed at one end and free at the other, and the rotation of the free end is desired, the unit moment applied there would cause a unit moment at every section, but this is not true for a beam supported at both ends or for a bar in a frame. Professor Burr's Equation (2), which he appears to think the correct form of the writer's Equation (4), by making $M_r = 1$, is in reality a totally different thing, for it gives the change of slope between two given points, while the writer's Equation (4) gives, as is clearly stated, the angular rotation at a given point. In a beam fixed at one end, his Equation (2) does, of course, give the angular rotation at the free end,

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but this is only because there is no rotation at the fixed end. His Equation (2), then, which he states gives the angular displacement of a point in the axis of a beam, only does so when the integration is extended from this point to some other point where the slope is not changed, and this point must first be found, unless its position is obvious, which is not generally the case.

The writer has made no analytical error, as Professor Burr claims. In his first illustration of what he considers this error, Professor Burr has perhaps been misled by the obvious typographical error in the writer's Equation (7), which should not have confused any one. In his second illustration of this error, Professor Burr finds his Equation (8), which is the same as the writer's Equation (4), for the case of a single beam in bending, and in which $d\ \alpha$ is, of course, the angular rotation only at the point where the unit moment is applied, and he then incorrectly uses this value as if it were the angular rotation at any point.

The writer's Equation (14), alluded to by Professor Burr, is of course the most general form of the Theorem of Three Moments only when the moment of inertia is constant. This is obvious, but perhaps it should have been stated in order that no one might have been misled.

To sum up, Professor Burr has evidently entirely failed to grasp the meaning or application of the principle. The fact, however, that he, who for so long has been an honored and leading member of the Profession, and at the head of one of our best engineering schools, was not only not familiar with the principle, but does not grasp it and denies its correctness when presented, should be sufficient to prove that the paper was needed, and quite a sufficient commentary on the remarks of Mr. Janni, whose discussion will now be considered.

Mr. Janni goes out of his way to find as much fault as he can with the principle and its presentation. Apparently, however, he has not succeeded in finding any ground for denying its correctness. He says it is "obsolete", not having yet perceived that a scientific principle, if correct, can never be obsolete. He says that "any good textbook will give some method of deriving this formula", but he wisely refrains from naming any book which does. He thinks that "any student in any engineering school should know" the principle, and that it is "inferior to modern methods". He then proceeds to attempt to show how much better he could solve structural problems, and begins by stating a mysterious "mathematical truth" which gives the angular rotation of the free end of an elastic body fixed at the other end, using what is called "elastic weight" and "elastic center", terms introduced by Professor Ritter, of Zurich, thirty-two years ago, which, according to Mr. Janni's ideas, should be "obsolete" by this time. He refrains from demonstrating his mathematical truth, but does not state whether this is because "any student in any engineering school" should know it,

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or because it is too advanced and modern. If he had approached the discussion in a scientific or even courteous spirit, instead of simply trying to find fault, he might have perceived and pointed out that his "mathematical truth" is most easily demonstrated by the writer's principle, of which that truth is merely a corollary or application; but if the principle is obsolete, perhaps the corollary is also. Referring to his Fig. 33, and using the writer's principle, if a unit moment is applied at the free end the moment at any section is unity, and if y is the distance from the force P (applied at the free end) to any point of the axis of the piece, the moment at that section due to P is Py. Hence, by Equation (4) of the writer's paper

$$\alpha = \int_0^l \frac{P y \, d\,l}{E\,I} = P \int_0^l \frac{y \, d\,l}{E\,I},$$

or P times the distance from P to the center of gravity of all the $\frac{d\,l}{I\,E}$'s times the sum of all the $\frac{d\,l}{E\,I}$'s, or, as he expresses it, $P\,G\,d$. This mysterious truth is, therefore, merely a corollary of the writer's principle of this particular case.

Now, it is always better to keep general principles in mind than corollaries, for the latter may be deduced at once if desired. In this way the memory will be relieved as much as possible, and any particular problem may be attacked in the manner best suited to the case.

It is easy now to take up the cases shown by Mr. Janni, and to show that they are either incorrectly solved by him or may be easily solved by the writer's principle, or other and well-known fundamental principles, without using elastic weight or any other conception, which may only complicate while giving the appearance of erudition. This is not said in derogation of the use of the "elastic weight", for no principle of science, and no corollary of one, ever becomes obsolete, but may at times be of use. The point is to emphasize the fundamentals.

Considering Mr. Janni's Case 1: This most elementary case of a beam of constant section fixed at one end is most easily solved by the area moment method, and the slope at the free end may be at once written down from the moment area, or $\frac{P\,l}{2}\,\frac{l}{E\,I}$. The writer's principle is precisely the same, since $M_x = 1$. Elastic weight, therefore, is unnecessary and also confusing here. If the section is not constant, the writer's Equation (7) may be at once written, without the last term, and $\alpha = P$ (sum of each $\frac{d\,x}{E\,I}$ multiplied by its x); while Mr. Janni would have us first find the d of the center of gravity of the

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$\frac{d\,x}{E\,I}$'s by finding the quantity in parentheses above, and dividing it by the sum of all the $\frac{d\,x}{E\,I}$'s, and then multiplying by this same sum. The elastic weight is here neither necessary nor desirable, but merely a complication, as "any student in any engineering school" can easily see. Treating Mr. Janni's second case by the writer's method, a left-handed unit moment applied at D would cause a unit moment at every section of the three parts, and no direct stress. The load, P, produces direct stresses as well as moments, but as the writer explained in his paper, the former are not to be considered. Resolving P at D into horizontal and vertical components, H and V, the moments produced by P are:

In DB, a moment varying from zero at D to Hh at B;
In BA, a moment varying from Hh at B to $(Hh - Vl)$ at A;
In AC, a moment varying from $(Hh - Vl)$ at B (causing compression on the left-hand fibers), to Vl at C (causing compression on the right-hand fibers).

The value of the angular rotation at D may at once be written, observing signs, as

$$E\,I\,\alpha = \frac{H\,h^2}{2} + H\,h\,l - \frac{V\,l^2}{2} + H\,h^2 - V\,h\,l - \frac{H\,h^2}{2}$$

$$= H\,h\,(l + h) - \frac{V\,l}{2}\,(l + 2\,h).$$

If the load P is under control it may sometimes be of use to recognize that, if it passes through the point G, α will be zero, but this is shown just as simply by the ratio of V to H given by the above equation, as by finding G.

Mr. Janni's Case 3 is the same as that illustrated by Fig. 33 and the writer has already shown that his principle gives the solution more simply and easily than Mr. Janni's method. Referring to his Case 4, if we observe what Mr. Janni's method really is, we shall see that it is inaccurate, and that so far as it is correct it is merely a "camouflaged" application of very old principles. To find the angular rotation of the bar B at the point 13 under a force F at 13, it is clear that the lower chord bars will be in compression and the upper chord bars in tension. Any shortening of the distance 1-3 will cause a rotation about the point 2, and if S is the stress in 1-3, the shortening will be $\frac{S\,l}{E\,A}$ and the angular rotation about 2 will be $\frac{S\,l}{E\,A\,r}$. For a load F at 13, the value

of S will be $\frac{F a_2}{r}$, if a_2 is the distance from 2 to F, hence the angular rotation will be $\frac{F l a_2}{E A r^2}$, and this rotation will affect all the structure beyond 2. The same is true for each chord bar, all rotations being in this case in the same direction; hence the total angular rotation of B will be $F \Sigma \frac{l a}{E A r^2}$, in which a is the distance of any point from F, and l the length of the chord bar opposite the joint, E and A its modulus of elasticity and area, and r its distance from the joint. If a force, $\frac{l}{E A r^2}$, is considered to act at the joint opposite l, this angular rotation is merely F (sum of moments of these forces about any point in the line of F). This sum may be found by the elementary principle of graphical statics, by drawing the string polygon for these loads, drawing through the point about which the moment is desired a vertical line (that is, prolonging F), and finding the intercept, y, between the end strings. The moment will be the pole distance times y, and by taking the pole distance as $\sum \frac{l}{E A r^2} = G$, then $G y$ is the moment, and this times F is the rotation.

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This is not modern, but is merely using Culmann's principle of finding moments graphically. One wonders that Mr. Janni did not consider this principle obsolete. Mr. Janni's application, however, is erroneous, because he neglects all the web members. In solid beams, the deformation due to shear is often or generally neglected because (1) it cannot be accurately determined, and (2) it is often or generally small. In trusses, the deformation due to shear may be determined as accurately as for the chords, and it may be considerable. Professor Ritter cites a case in which the deflection due to web members was 41% of the total. There is no excuse here for neglecting the web. By the writer's principle, the stress in every member would be found; the unit moment being applied at 13, the stress in every chord piece due to it would be $\frac{1}{r}$ and the stress due to F would be $F a \div r$. The web would be similarly included. The summations may be made more quickly, easily, and much more accurately than by Mr. Janni's method, which does not and cannot well include the web, and which is therefore distinctly inferior. This example well illustrates the danger of using methods that one does not thoroughly understand, and the importance of grasping and adhering to fundamental principles.

The writer cordially thanks Messrs. Hudson, Mensch, Eddy, Cain, and Maney for their interesting contributions, which call for no specific comment.

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To sum up, it is no doubt true, as pointed out by Messrs. Church, Eddy, Cain, and Mensch, that the writer's principle is implicitly contained in Castigliano's work equation. So was the latter implicitly contained in the law of the conservation of energy, and the principles of dynamics in Newton's laws of motion. The writer, however, has not found in Castigliano any statement of this principle of rotation. He admits that the principle as applied to beams was known in Europe, but he has not found it explained in any American book, neither in Church, Molitor, Turneaure, Hudson, nor any other. Further, he still fails to see that any author, anywhere, has clearly shown its general applicability, to trusses as well as to beams, or even stated it, as a principle analogous to the deflection principle of Maxwell. Hence, he still thinks it may be called new.

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Paper No. 1439

PRESSURES IN PENSTOCKS CAUSED BY THE GRADUAL CLOSING OF TURBINE GATES

By Norman R. Gibson, M. Am. Soc. C. E.

With Discussion by Messrs. Otto V. Kruse, Eugene E. Halmos, R. D. Johnson, Minton M. Warren, T. Noble Anderson, Ford Kurtz, William P. Creager, and Norman R. Gibson.

Synopsis.

This paper shows how the rise of pressure caused by the gradual closing of turbine gates may be determined from Professor Joukovsky's theory of maximum water-hammer. A solution of the problem by the trial-and-error method of arithmetic integration is first given, and then formulas are derived that cover any governor time and any relation between governor stroke and gate movement. Two factors which affect the rise of pressure are taken into consideration: (1) the elasticity of water and of the walls of the penstock; and (2) the effect of the net head on the phenomena which occur during the closing of the gates. Previous writers have submitted formulas which neglect one or other of these two factors, with the result that they do not give correct results in all cases. The limitations of such formulas are determined by the solution of the problem herein submitted.

Introduction.

The purpose of this paper is to show how the excess pressure in penstocks caused by the gradual closing of turbine gates may be determined from Professor Joukovsky's theory of water-hammer. It will be assumed that the theory of pressure waves, their amplitudes, and speeds of propagation, as formulated by him and proved by his experiments, may be accepted as correct.

At the risk of wearying the reader who is familiar with Joukovsky's work, it is necessary, in connection with what is to follow, to summarize as briefly as possible the principles demonstrated a number of years ago by this distinguished Russian. For a partial translation of his work, the reader is referred to Miss O. Simin's paper entitled "Water Hammer"* which should be examined carefully by every student of this subject.

Joukovsky's experiments, made in 1898 at Moscow, were confined to the instantaneous stopping of the flow of water in long pipes. By his experiments he was able to prove the soundness of his analytical determination of the maximum rise of pressure that would occur when the flow of water in a pipe was suddenly arrested. The casual thinker at first would imagine that—as force is equal to the product of mass by acceleration—an infinite pressure would be produced in a pipe if the water flowing in it were stopped instantaneously. On second thought, he would realize that neither the water column nor the walls of the pipe are rigid, and, therefore, the pressure caused by the shock of stopping the flow suddenly is relieved by the slight compression of the water and the expansion of the walls of the pipe. It was the effect of these two factors that was determined by Professor Joukovsky. He showed that the shock pressure is transmitted along the column of water in the pipe in waves similar to sound waves; and that the shock pressure is proportional to the destroyed velocity of flow and to the speed of propagation of the pressure waves. This speed depends on the compressibility of water, on the elasticity of the materials of the pipe, and on the ratio of the thickness of the walls of the pipe to its diameter. In other words, if the speed of the pressure wave is known, the maximum pressure produced (called water-hammer) by instantane-

* *Proceedings,* Am. Water Works Assoc., 1904, p. 341.

ously stopping water flowing in a pipe at any velocity may be calculated.

Joukovsky's formula for water-hammer is:

$$h = \frac{a\ V}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where h = excess pressure, in feet;
V = velocity of flow in the pipe, in feet per second;
g = gravitational unit, in feet per second per second;
and a = velocity of the pressure wave, in feet per second, which is determined by the formula:

$$a = \frac{12}{\sqrt{\frac{W}{g}\left\{\frac{1}{k} + \frac{d}{E\ e}\right\}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where W = weight of a cubic foot of water, in pounds;
g = gravitational unit, in feet per second per second;
k = voluminal modulus of water, in pounds per square inch;
d = diameter of pipe, in inches;
e = thickness of pipe walls, in inches;
E = modulus of elasticity of material of pipe walls, in pounds per square inch.

Joukovsky showed also that the shock pressure is transmitted along the pipe with constant intensity and "at constant velocity, which seems to be independent of the intensity of the shock."

"The speed of propagation of the pressure wave remains the same, whether the shock is caused by arresting the flow of a column of water moving in a pipe, or by suddenly changing the pressure in the column of water (flowing or standing) in any part and by any other means."

"If the water column continues flowing, such flow exerts no noticeable influence upon the shock pressure. In a pipe from which water is flowing, the pressure wave is reflected from the open end of the pipe, in the same way as from a reservoir with constant pressure."

"The phenomenon of periodical vibration of the shock pressure is completely explained by the reflection of the pressure wave from the ends of the pipe, *i. e.*, from the gate and from the origin [of the pipe]."*

Pressure waves, after traveling up the penstock to the origin or point of relief and back to the gate, are reflected and transmitted again

* Quotations are from "Water Hammer," by Miss O. Simin.

over the same course, as waves of rarefaction or sub-normal pressure. In this manner pressure waves, alternately super-normal and sub-normal, travel up and down the penstock until damped out by friction. During the time taken by the wave to traverse its course from gate to origin the pressure at the gate remains at its full value, either super-normal or sub-normal, as the case may be.

Briefly stated, then, the premises are: that when water flowing in a pipe is suddenly arrested, certain pressure waves, the characteristics of which are known, are produced and propagated along the pipe at constant speed and constant magnitude, and that the speed and magnitude of these waves may be calculated for any given conditions.

Fundamental Equations.

It is the writer's intention to apply herein this theory of pressure waves to the phenomena which occur when the gates of a turbine at the end of a penstock are gradually closed. The damping effect of friction on the pressure waves will be neglected during the time of closure. The variable velocity of the pressure wave due to the difference in density of the water at the top and bottom of the penstock will also be neglected. For the sake of simplicity, it will be assumed that the gate opening is closed uniformly from full open to shut by a governor which moves the gates at uniform velocity from the beginning of its stroke to the end, and that the area of gate opening is directly proportional to the amount of gate movement. It may be stated here, parenthetically, that the resulting formulas may be modified, easily, to suit any method of gate closure, whether the speed of closing and the relation of governor movement to area of gate opening is uniform or variable.

When the gates of a turbine are closed gradually the velocity of the water in the penstock is reduced to zero and the pressure in the penstock rises. It is frequently assumed that the reduction in velocity takes place uniformly, but the rise of pressure, which commences immediately after the gates begin to move, increases the velocity of discharge through the gate opening, and, during the early part of the gate movement, tends to diminish the rate at which the flow of the water is retarded. This variable rate of retardation, during the time the gates are being closed, has an important bearing on the resulting rise of pressure, and it is necessary to take it into consideration by

determining the relation between the velocity of flow and the pressure in the penstock. This relation may be expressed by the equation:

$$V_0 = B_0 \sqrt{H_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where V_0 = initial velocity, in feet per second, of the water in the penstock before shut down;

H_0 = normal net head, in feet;

and B_0 = a number representing the gate opening. The value of B_0 is best determined from the known value of V_0 and H_0, but, in fact, B_0 is the ratio of the area of the gate opening to the area of the penstock multiplied by $\sqrt{2g}$ times the coefficient of discharge of the gate opening.

At any time during the closing of the gates the relation between V, B, and H would be expressed by the general formula, $V = B \sqrt{H}$. If the gates are closed in the time, T, and t is the time from the beginning of the stroke to any time before the end of the stroke, the value of B_t (that is, B at the end of the time, t) for uniform closing, would be $\left(1 - \frac{t}{T}\right) B_0$. During the time, t, the pressure in the penstock has risen an amount, h_t, so that the net head, H_t (that is, H at the end of the time, t) would be equal to $H_0 + h_t$. Therefore, the expression for the value of V_t (that is V at the end of the time, t) is:

$$V_t = \left(1 - \frac{t}{T}\right) B_0 \sqrt{H_0 + h_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Calculation by Arithmetic Integration.

Before proceeding with the analytical determination of h_t, it will perhaps make the work clearer to show first, by a numerical example, how h_t may be otained by the trial-and-error method of arithmetic integration. Assume that the gate, instead of being moved in a continuous uniform manner, is closed by a series of small instantaneous movements with a slight pause between each movement. Each little movement of the gate would destroy instantaneously a small part, ΔV, of the velocity, V_0, and, since this part of the velocity is destroyed instantaneously, the rise of pressure, according to Joukovsky, would be $h = \frac{a \Delta V}{g}$. When the first instantaneous movement has taken place, let

a pause of time, t, elapse before the next movement. Then $\Delta V = V_0 - V_t$, and $h_t = \frac{a}{g}(V_0 - V_t)$.

By these equations and Equation (4) a numerical example may now be solved, and at the same time there will be explained other interesting phenomena caused by the closing of the gates and the pressure waves produced thereby. The analytical work may then be more readily understood.

Let $L = 820$ ft.;

$V_0 = 11.75$ ft. per sec.;

$H_0 = 165$ ft.;

$T = 2.1$ sec. (For convenience, T has been chosen an even multiple of $\frac{2L}{a}$);

$a = 4\,680$ ft. per sec. (This value of a is chosen simply because this example was worked out by the writer for a pipe in a tunnel and concreted in. The expansion of the pipe walls, therefore, was neglected. For any condition, a may be obtained by Equation (2)).

Since $V_0 = B_0 \sqrt{H_0}$, then $B_0 = 0.91476$.

Assume that the gates are closed in 24 successive instantaneous movements. The time elapsing between each movement would then be 0.0875 sec. After the first of these movements had taken place the gates would have been closed one-twenty-fourth of their opening, and the number representing the gate opening would have been reduced by one-twenty-fourth of its value, that is, $\frac{0.91476}{24} = 0.038115$. At each of the successive movements the value of B is reduced by the same amount, as the gate motion is assumed to be uniform. It will not be necessary, however, to use more than the first three significant figures, and the work may be done on the slide-rule. In this example the recovery of the friction head in the penstock will be neglected.

From the foregoing may now be written the first three columns of Table 1, and the first line of Columns 4 and 5. The table may then be completed as follows: Assume a trial reduction in velocity, caused by the initial instantaneous movement of the gate, and set the figure down in Column 5 under the value of V_0 and subtract it from V_0, placing

TABLE 1.—By Arithmetic Integration.

Data: $L = 820$ ft. $V_0 = 11.75$ ft. per sec. $H_0 = 165$ ft. $T = 2.1$ sec. $a = 4\,680$ ft. per sec. Friction neglected.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Interval.	Time, t.	Gate, B.	Head, H.	Velocity, V.	$145\,\Delta V$, Δh.	$\Sigma(\Delta h)$, h_t.
0	0.0000	0.91476	165.00	11.75	.....	.....
		0.03811		0.085	12.32	.....
¼	0.0875	0.87665	177.32	11.665		12.32
		0.03811		0.095	13.78	
½	0.1750	0.83854	191.10	11.570		26.10
		0.03811		0.10	14.50	
¾	0.2625	0.80043	205.60	11.47		40.60
		0.03811		0.112	16.20	
1	0.3500	0.76232	221.80	11.358		56.80
		0.03811		0.258	37.40	
1¼	0.4375	0.72421	234.56	11.10		69.56
		0.03811		0.29	42.10	
1½	0.5250	0.68610	249.10	10.81		84.10
		0.03811		0.30	43.50	
1¾	0.6125	0.64799	263.60	10.51		98.60
		0.03811		0.33	47.80	
2	0.7000	0.60988	279.00	10.18		114.00
		0.03811		0.43	62.30	
2¼	0.7875	0.57177	291.14	9.75		126.14
		0.03811		0.47	68.20	
2½	0.8750	0.53366	302.70	9.28		137.70
		0.03811		0.48	69.60	
2¾	0.9625	0.49555	314.30	8.80		149.30
		0.03811		0.53	76.80	
3	1.0500	0.45744	327.90	8.27		162.90
		0.03811		0.58	84.00	
3¼	1.1375	0.41933	337.46	7.69		172.46
		0.03811		0.61	88.40	
3½	1.2250	0.38111	346.10	7.08		181.10
		0.03811		0.62	90.00	
3¾	1.3125	0.34300	354.90	6.46		189.90
		0.03811		0.67	97.10	
4	1.4000	0.30489	361.60	5.79		196.60
		0.03811		0.68	98.60	
4¼	1.4875	0.26678	366.60	5.11		201.64
		0.03811		0.70	101.50	
4½	1.5750	0.22867	371.10	4.41		206.10
		0.03811		0.715	103.80	
4¾	1.6625	0.19056	376.10	3.695		211.10
		0.03811		0.730	106.00	
5	1.7500	0.15245	378.30	2.965		213.30
		0.03811		0.730	106.00	
5¼	1.8375	0.11434	380.66	2.235		215.66
		0.03811		0.745	108.00	
5½	1.9250	0.07623	382.70	1.490		217.70
		0.03811		0.745	108.00	
5¾	2.0125	0.03812	381.90	0.745		216.90
				0.745	108.00	
6	2.1000	0.0	381.70	0.0		216.70

the difference immediately underneath. This trial figure is ΔV, and is assumed to be destroyed instantaneously by the first movement of the gate. A pressure wave, Δh, of magnitude, $\frac{a}{g}\Delta v = \frac{4\,680}{32.2}(\Delta V) = 145\,\Delta V$, is therefore started up the pipe. The product of $145\,\Delta V$ is set down

in Column 6 opposite ΔV. In Column 7 is recorded the algebraic sum of the values of Δh. Having obtained the figure in Column 7, it is added to the net head, $H_0 = 165$, and the sum is set down in the next line lower in Column 4. The result must now be checked, to see that $B\sqrt{H} = V$, where B is 0.8766 and H and V have the values recorded in their respective columns opposite B. If the relation is not satisfied, a new trial value of ΔV must be chosen, and the operations repeated until a check is obtained. After trial, the initial value of ΔV was found to be 0.085. Proceeding in this way, the rise of pressure at the end of 0.35 sec. is found to be 56.80 ft. This rise has taken place in four successive jumps.

At this point it becomes necessary to trace the course of the pressure wave started up the penstock by the initial movement of the gate. This wave has a velocity of 4 680 ft. per sec., and travels at that rate toward the forebay or origin of the penstock; after arriving at the origin it is reflected and returns to the gate at the same velocity. The distance from gate to forebay and return is 1 640 ft., so that the pressure wave takes 0.35 sec. to cover this distance. On its arrival at the gate it is reflected immediately as a wave of sub-normal pressure, and commences its journey again from gate to forebay and back. At the instant the wave becomes sub-normal, however, the gate is given one of its instantaneous closing movements, causing a further reduction in the velocity of the water flowing in the penstock and the consequent rise of pressure incident thereto. Thus, at this instant, two factors have to be taken into consideration: the rise of pressure caused by the fifth little instantaneous movement of the gate and the fall in pressure caused by the change from super-normal to sub-normal of the pressure wave produced by the first or initial movement of the gate which occurred 0.35 sec. before.

By trial-and-error the velocity that has been destroyed by the fifth movement of the gate is found to be 0.258 ft. per sec., and the result is checked as follows. Multiplying 0.258 by 145, the magnitude of the resulting pressure wave is 37.4 ft., and this added to 56.80 would make the total excess pressure existing equal to 94.20 ft., were it not for the fact that the initial wave has returned to the gate and become sub-normal. The amount of the initial wave, as shown by the second line of Table 1, is 12.32, and since it not only falls to zero but passes below zero to a sub-normal pressure of equal amount, there must be subtracted

twice 12.32 from 94.20, making the net excess pressure existing at that instant 69.56 ft. Adding this to the initial net head, $H_0 = 165$, the value of V is checked as before by the formula, $V = B\sqrt{H}$.

Proceeding then as before, and remembering that the pressure rise caused by the sixth movement of the gate is reduced by the fall to sub-normal of the wave produced by the second movement of the gate, and the pressure rise of the seventh movement is reduced by the wave produced by the third movement, and so on, there may be calculated the successive increments of pressure.

For convenience, the time required by the pressure waves to travel from the gate to the origin of the pipe and return to the gate again will be called one interval. The interfering waves referred to in the foregoing paragraph are then always one interval apart.

After the eighth movement of the gate has taken place, it is found that the net excess pressure has reached 114.00 ft. At the instant the ninth movement takes place it must be noted that the pressure wave produced by the first movement of the gate, which has traveled, during the first interval, from the gate to the origin and back as a wave of super-normal pressure, and during the second interval over the same course as a wave of sub-normal pressure, has now returned to the gate and, again becoming super-normal, is again reflected and commences its journey from gate to origin and back. Thus, at the ninth movement of the gate, there must be taken into consideration the pressure wave caused by the instantaneous destruction of the velocity at that instant, the sub-normal wave due to the fifth movement, and the super-normal wave due to the first movement. After adding to the excess pressure existing at the end of the eighth movement, the rise of pressure caused by the ninth movement, there must be subtracted twice the pressure caused by the fifth movement, and there must be added twice the pressure caused by the first movement, or, in other words, there is subtracted twice the difference between the fifth and first waves.

By thus keeping in mind the position of the wave propagated by each movement of the gate, Table 1 may be completed, and the resulting maximum rise of pressure is found to be 217.70 ft.

Fig. 1 is a series of graphical diagrams showing the magnitude of the pressure waves caused by the successive instantaneous movements of the gate. A separate diagram is drawn for each movement, pressure being represented by the ordinates and time by the abscissas

The change from super-normal to sub-normal and *vice versa* is shown at the end of each interval of time. Fig. 2 is the excess pressure-time curve plotted from the figures in Column 7 of Table 1, or, what amounts to the same thing, from the algebraic sum of the pressure waves shown in Fig. 1.

Table 2 shows the method of determining the results when the recovery of friction in the penstock is included, an example being selected where the friction head is an appreciable quantity. The

TABLE 2.—By Arithmetic Integration.

Data: $L = 6\,337$ ft. $V_0 = 15.055$ ft. per sec. $H_0 = 1\,260$ ft. $T = 69.5$ sec. $a = 3\,647$ ft. per sec. Friction head $h_f = 81$ ft.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Interval.	Gate,* B.	Head, H.	Velocity, V.	$113\,\Delta V$, Δh.	$\Sigma\,\Delta h$.	$3\,574\,V^2$, h_f.	Δh_f.	$\Sigma(\Delta h + \Delta h_f)$, h_t.
0	0.4241	1260.0	15.055		....	81.0	....	....
			0.155	17.5				
1	0.4168	1279.2	14.90		17.5	79.3	1.7	19.2
			0.29	32.8				
2	0.4084	1279.8	14.61		15.3	76.5	4.5	19.8
			0.28	31.7				
3	0.3995	1284.0	14.33		16.4	73.4	7.6	24.0
			0.33	37.3				
4	0.3895	1291.8	14.00		20.9	70.1	10.9	31.8
			0.37	41.8				
5	0.3784	1295.5	13.63		20.9	66.4	14.6	35.5
			0.42	47.5				
6	0.3657	1305.3	13.21		26.6	62.3	18.7	45.3
			0.49	55.4				
7	0.3513	1311.8	12.72		28.6	58.0	23.0	51.8
			0.53	59.8				
8	0.3354	1319.0	12.19		31.0	53.0	28.0	59.0
			0.60	67.8				
9	0.3180	1329.8	11.59		36.8	48.0	33.0	69.8
			0.65	73.5				
10	0.2991	1334.9	10.94		36.7	42.8	38.2	74.9
			0.72	81.3				
11	0.2781	1348.2	10.22		44.6	37.4	43.6	88.2
			0.84	94.8				
12	0.2544	1359.7	9.38		50.2	31.5	49.5	99.7
			0.92	104.0				
13	0.2285	1369.2	8.46		53.8	25.6	55.4	109.2
			0.97	109.5				
14	0.2009	1376.6	7.49		55.7	20.1	60.9	116.6
			1.07	121.0				
15	0.1719	1391.6	6.42		65.3	14.7	66.3	131.6
			1.15	130.0				
16	0.1411	1395.8	5.27		64.7	9.9	71.1	135.8
			1.21	136.8				
17	0.1083	1407.2	4.06		72.1	5.9	75.1	147.2
			1.33	150.1				
18	0.0725	1416.3	2.73		78.0	2.7	78.3	156.3
			1.365	154.2				
19	0.0363	1416.5	1.365		76.2	0.7	80.3	156.5
			1.365	154.2				
20	0.	1419.0	0.		78.0	0.	81.0	159.0

* Non-uniform gate motion.

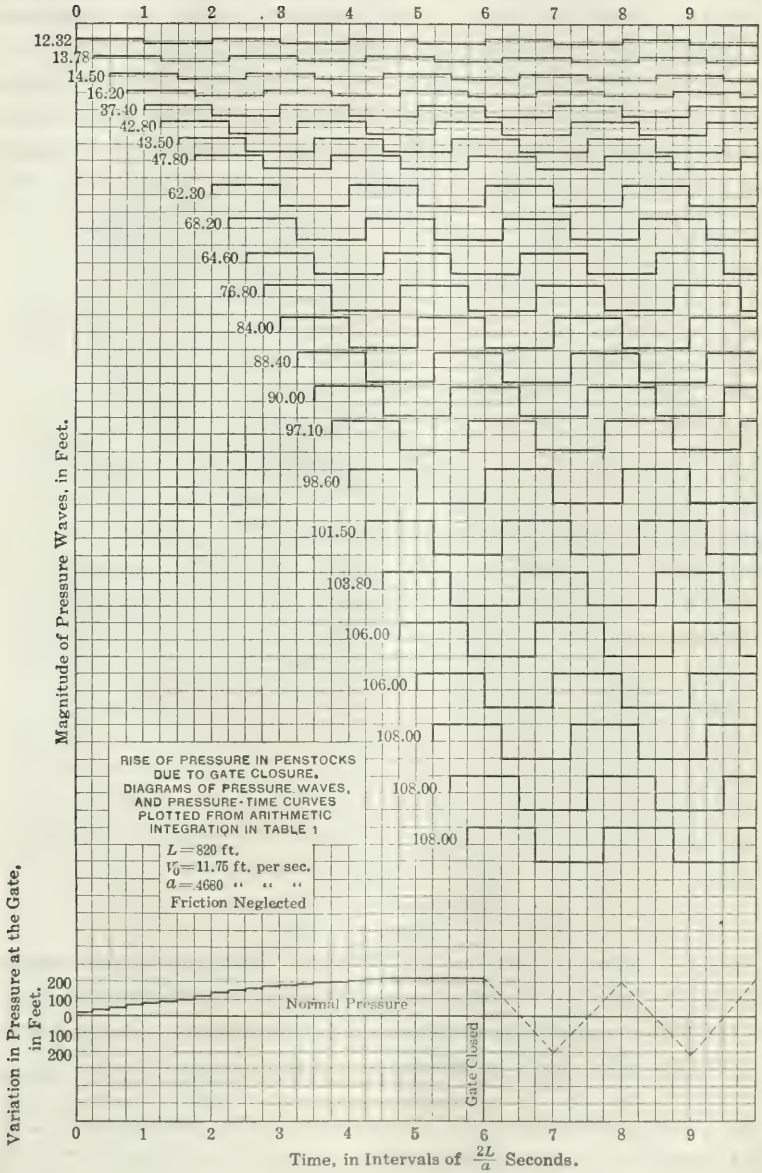
0 1 2 3 4 5 6 7 8 9
12.32
13.78
14.50
16.20
37.40
42.80
43.50
47.80
62.30
68.20
64.60
76.80
84.00
88.40
90.00
97.10
98.60
101.50
103.80
106.00
106.00
108.00
108.00
108.00
Magnitude of Pressure Waves, in Feet.
RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE, DIAGRAMS OF PRESSURE WAVES, AND PRESSURE-TIME CURVES PLOTTED FROM ARITHMETIC INTEGRATION IN TABLE 1
L = 820 ft.
V_0 = 11.75 ft. per sec.
a = 4680 " " "
Friction Neglected
Normal Pressure
Gate Closed
200
100
0
100
200
Variation in Pressure at the Gate, in Feet.
0 1 2 3 4 5 6 7 8 9
Time, in Intervals of $\frac{2L}{a}$ Seconds.

FIG. 1.

operations are almost identical with those just described, up to Columns 6 and 7. To avoid reference to the foregoing, however, the formation of Table 2 will be described from the beginning. In the first line, opposite B = 0.4241, set down in Columns 3, 4, and 7, the known values of H_0, V_0, and h_f (friction head). Next, assume a trial reduction in velocity, caused by the initial instantaneous movement of the gate, and set the figures down in Column 4 under the value of V_0, and

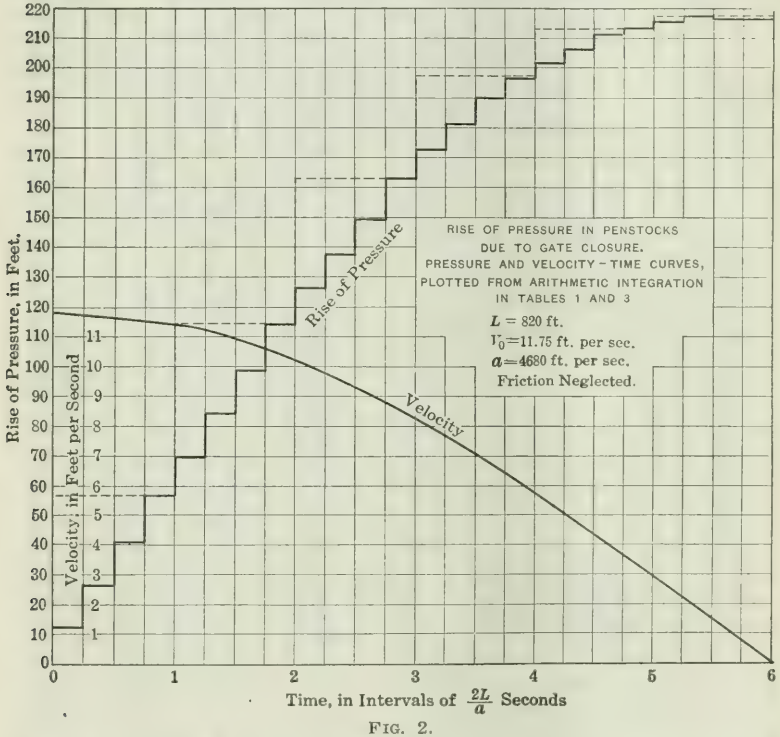
Rise of Pressure, in Feet.
Velocity, in Feet per Second
Time, in Intervals of $\frac{2L}{a}$ Seconds
Rise of Pressure
Velocity
RISE OF PRESSURE IN PENSTOCKS
DUE TO GATE CLOSURE.
PRESSURE AND VELOCITY – TIME CURVES,
PLOTTED FROM ARITHMETIC INTEGRATION
IN TABLES 1 AND 3
L = 820 ft.
V_0=11.75 ft. per sec.
a=4680 ft. per sec.
Friction Neglected.

FIG. 2.

subtract it from V_0, placing the difference immediately underneath. This trial figure is ΔV, and is assumed to be destroyed instantaneously by the first movement of the gate. A pressure wave, Δh, of magnitude $= \frac{(\Delta V)\, a}{g} = (\Delta V)\, \frac{3647}{32.2} = 113\ \Delta V$, therefore, is started up the pipe. The product of 113 ΔV is set down in Column 5 opposite ΔV. In Column 6 is recorded the algebraic sum of the values of Δh. In Column 7 the total friction head, due to the velocity shown in Column 4, is set down, and, in Column 8, the friction head recovered at each

operation is shown. For convenience, h_f may be made equal to $F\ V^2$, in which F is a coefficient obtained from the known values at the beginning. In the example, $F = 0.3574$. Column 9 shows the sum of the opposite items in Columns 6 and 8. Having obtained the figure in Column 9, it is added to the net head, $H_0 = 1\,260$, and the sum is set down in the next lower line in Column 3. The result must now be checked to see that $B \sqrt{H} = V$, where B is now 0.4168, and H and V are the values opposite. If the relation is not satisfied, a new trial value of ΔV must be chosen, and the operations repeated until a check is obtained. After trial, the initial value of ΔV was found to be 0.155. The operations for obtaining the figures on the third line are not quite the same as those just described, because, after assuming the next trial value of ΔV, and multiplying it by 113, it must be remembered that the resulting pressure at the gate is reduced by the return of the first wave, which has traveled up to the forebay and back, and now changes to sub-normal and repeats the journey. The time at which the gate is given its second movement has been selected purposely to coincide with the return of the first wave. The item opposite Interval 2, in Column 6, therefore, is the difference between the first two figures in Column 5. The other operations are similar to those already described, and the resulting figures in Columns 3 and 4 are checked similarly with the value of $B = 0.4084$. The succeeding lines are filled in by the same process, always keeping in mind the return of the preceding waves, and whether they change to sub-normal or super-normal. A little study will disclose the fact that the figure in the second line of Column 6 may be obtained by subtracting the figure in the first line of Column 6 from the figure in the second line of Column 5. Similarly, the figure in the third line of Column 6 may be obtained by subtracting the figure in the second line of Column 6 from the figure in the third line of Column 5, and so on.

At this point it is interesting to repeat the trial-and-error work for the foregoing example worked out in Table 1, but using only 6 instantaneous movements of the gate instead of 24. The results are shown in Table 3, and the pressure-time curve in dotted lines in Fig. 2. From these it will be noted that the total rise of pressure is the same as that obtained by the calculations for 24 movements, and, moreover, the resulting pressure at the end of each interval is the same in both cases. Although six movements of the gate, or one movement to each

TABLE 3.—By Arithmetic Integration.
For Data, see Table 1.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Interval.	Time, T.	Gate, B.	Head, H.	Velocity, V.	$145\ \Delta\ V$, $\Delta\ h$.	$\Sigma\ (\Delta\ h)$, h_t.
0	0.0	0.91476	165.00	11.75		.....
				0.392	56.8	
1	0.35	0.762	221.8	11.358		56.8
				1.178	171.0	
2	0.70	0.608	279.2	10.18		114.2
				1.91	277.0	
3	1.05	0.457	327.8	8.27		162.8
				2.48	360.0	
4	1.40	0.304	362.20	5.79		197.2
				2.825	410.0	
5	1.75	0.1525	377.8	2.965		212.8
				2.965	430.0	
6	2.10	0.0	382.2	0.00		217.2

interval of time, are sufficient to determine the pressure rise at the end of each interval, it requires the larger number of movements to obtain intermediate points on the pressure-time curve. If a still greater number of movements is taken, the increments in pressure rise become smaller, and, in the limit, the stepped diagrams similar to Fig. 2 would become a series of smooth curves from the beginning to the end of each interval. The diagram, however, would not necessarily form a smooth curve from the beginning to the end of the closing time, because cusps or changes of curvature at the end of each interval result from the action of the pressure waves in changing at that instant from super-normal to sub-normal or *vice-versa*. When the duration of closure is short, the change of curvature in the diagram at the end of each interval is frequently very apparent; but, when the duration of closure is long, the changes of curvature in many cases cannot be detected by the eye. These changes of curvature at the end of the intervals make it difficult to formulate the integration which can be performed so easily by the trial-and-error work already explained.

It is possible, however, to obtain a series of equations, one for each interval of the closing time, that constitute a direct mathematical solution of the problem, without recourse to trial-and-error methods. The foregoing example has been explained in order that the analytical work may be more easily understood, and in order that the method of tracing the course of the pressure waves and keeping track of their periodic changes may be kept clearly in mind. More particularly, it should

be borne in mind that though a formula may be written for the pressure rise in the first interval, using only the known quantities existing before the shut-down, the formulas for the pressure rise in any succeeding interval will involve, as one of the known quantities, the value of the pressure rise at the end of the preceding interval. It thus becomes necessary, when calculating the pressure rise at any time during, or at the end of, the closing of the gate, to obtain first the amount of the pressure rise at the end of each preceding interval. Moreover, if it is desired to determine the pressure rise at any time between the beginning and end of any interval—that is, at any fractional part of an interval—not only is it necessary to determine the pressure rise at the end of each preceding interval, but also the pressure rise at the same fractional part of each preceding interval. This also applies in determining the excess pressure-time curve when the duration of closure is not exactly a whole number of intervals.

NOMENCLATURE.

L = length of penstock, in feet;

a = velocity of pressure wave, in feet per second;

g = acceleration due to gravity, in feet per second per second;

$\frac{2L}{a}$ = one interval of time;

T = governor time, in seconds, *i. e.*, total duration of gate closure;

$T_1, T_2, T_3, \ldots T_n$ = time at the end of the 1st, 2d, 3d, ... nth interval;

$t_1, t_2, t_3, \ldots t_n$ = any time during the 1st, 2d, 3d, ... nth interval;

n = any number of intervals; the final interval in the time, T, need not be complete;

H_0 = normal net head, in feet;

$h_1, h_2, h_3, \ldots h_n$ = excess pressure above normal, in feet, existing at the end of the 1st, 2d, 3d, ... nth interval, *i. e.*, at the times, $T_1, T_2, T_3, \ldots T_n$;

$h_{t_1}, h_{t_2}, h_{t_3}, \ldots h_{t_n}$ = excess pressure above normal, in feet, at any time during the 1st, 2d, 3d, ...nth interval, *i. e.*, at the times, $t_1, t_2, t_3, \ldots t_n$;

V_0 = initial velocity, in feet per second, of the water in the penstock before shut-down.

$V_1, V_2, V_3, \ldots V_n$ = velocity, in feet per second, of the water in the penstock at the end of the 1st, 2d, 3d, ... nth interval;

$V_{t_1}, V_{t_2}, V_{t_3}, \ldots V_{t_n}$ = velocity, in feet per second, of the water in the penstock at any time, $t_1, t_2, t_3, \ldots t_n$, during the 1st, 2d, 3d, ... nth interval;

$B_0 = \sqrt{\frac{V_0}{H_0}}$ = a number representing the gate opening;

$$S_{t_1} = \left(\frac{a}{g}\right)^2 \left(1 - \frac{t_1}{T}\right)^2 B_0^2 \qquad R_0 = \frac{a}{g} V_0$$

$$S_{t_2} = \left(\frac{a}{g}\right)^2 \left(1 - \frac{t_2}{T}\right)^2 B_0^2 \qquad R_{t_1} = h_1 + \frac{a}{g} V_1 - C_{t_1}$$

$$S_{t_n} = \left(\frac{a}{g}\right)^2 \left(1 - \frac{t_n}{T}\right)^2 B_0^2 \qquad R_{t_2} = h_2 + \frac{a}{g} V_2 - C_{t_2}$$

$$C_{t_1} = 2\frac{a}{g}(V_0 - V_{t_1}) \qquad R_{t_{n-1}} = h_{n-1} + \frac{a}{g} V_{n-1} - C_{t_{n-1}}$$

$$C_{t_2} = 2\frac{a}{g}(V_1 - V_{t_2}) - C_{t_1} \qquad Z_1 = \frac{\left(\frac{a}{g}\right)^2 S_{t_1}}{\left(\frac{a}{g}\right)^2 + S_{t_1} F}$$

$$C_{t_n} = 2\frac{a}{g}(V_{n-1} - V_{t_n}) - C_{t_{n-1}} \qquad Z_2 = \frac{\left(\frac{a}{g}\right)^2 S_{t_2}}{\left(\frac{a}{g}\right)^2 + S_{t_2} F}$$

F = a friction factor such that $F V^2$ = total loss, in feet of head, for velocity, V;

$$Z_n = \frac{\left(\frac{a}{g}\right)^2 S_{t_n}}{\left(\frac{a}{g}\right)^2 + S_{t_n} F}$$

Time is always to be measured from the beginning of the gate movement. Thus, the time, t_2, is the time from the beginning of the stroke to some time between the first and third intervals. The successive values of t_1, t_2, etc., must always be one interval apart.

First: Derivation of Formulas with Friction Neglected.—Joukovsky's formula for water-hammer, as already stated in Equation (1), is:

$$h = \frac{a V}{g}$$

During slow closing of the gate, $d\,h = \frac{a}{g}\,d\,V$.

At the beginning of the closing time, as already stated in Equation (3),

$$V_0 = B_0 \sqrt{H_0}$$

and, in general, $$V = B \sqrt{H_0 + h}.$$

For the first interval:

$$\int_0^{h_{t_1}} d\,h = \frac{a}{g} \int_{V_{t_1}}^{V_0} d\,V$$

where h_{t_1} = pressure rise at any time, t_1, during the first interval, and V_{t_1} = velocity at the time, t_1, during the first interval.

Integrating, then:

$$h_{t_1} = \frac{a}{g}\,(V_0 - V_{t_1}) \qquad (5)$$

Since V_{t_1} must be proportional to the gate opening multiplied by the square root of the head, then, for uniform gate motion:

$$V_{t_1} = \left(1 - \frac{t_1}{T}\right) B_0 \sqrt{H_0 + h_{t_1}} \qquad (6)$$

Substituting this value of V_{t_1} in the equation for h_{t_1}, the latter becomes:

$$h_{t_1} = \frac{a}{g}\,V_0 - \frac{a}{g}\left(1 - \frac{t_1}{T}\right) B_0 \sqrt{H_0 + h_{t_1}}$$

For simplicity, substitute the symbol, R_0, in place of $\frac{a}{g}\,V_0$, and the symbol, $\sqrt{S_{t_1}}$, for $\frac{a}{g}\left(1 - \frac{t_1}{T}\right) B_0$.

Thus: $h_{t_1} = R_0 - \sqrt{S_{t_1}\,(H_0 + h_{t_1})}$

Squaring: $(h_{t_1} - R_0)^2 = S_{t_1}\,(H_0 + h_{t_1})$

Expanding: $h_{t_1}^2 - 2\,h_{t_1}\,R_0 + R_0^2 = S_{t_1}\,H_0 + S_{t_1}\,h_t$

Collecting: $h_{t_1}^2 - h_{t_1}\,(S_{t_1} + 2\,R_0) + R_0^2 - S_{t_1}\,H_0 = 0$

Solving: $$h_{t_1} = \frac{1}{2}\left\{(S_{t_1} + 2\,R_0) \pm \sqrt{(S_{t_1} + 2\,R_0)^2 - 4\,R_0^2 + 4\,S_{t_1}\,H_0}\right\}$$

Simplifying: $$h_{t_1} = \frac{1}{2}\left\{(S_{t_1} + 2\,R_0) \pm \sqrt{S_{t_1}\,(S_{t_1} + 4\,R_0 + 4\,H_0)}\right\} \qquad (7)$$

For the second interval, an examination of Table 1 will show that

$$\int_0^{h_{t_2}} d\,h = \frac{a}{g} \int_{V_{t_2}}^{V_1} d\,V + h_1 - 2\,\frac{a}{g} \int_{V_{t_1}}^{V_0} d\,V$$

Where h_{t_2} = pressure rise at any time, t_2, during the second interval;
V_{t_2} = velocity at the time, t_2;
V_1 = velocity at the end of the first interval;
h_1 = pressure rise at the end of the first interval;
V_{t_1} = velocity at the time, t_1, which must be exactly one interval before the time, t_2.

Integrating: then $h_{t_2} = \frac{a}{g}(V_1 - V_{t_2}) + h_1 - 2\frac{a}{g}(V_0 - V_{t_1})$....(8)

Substituting C_{t_1} for $2\frac{a}{g}(V_0 - V_{t_1})$

$$h_{t_2} = \frac{a}{g}(V_1 - V_{t2}) + h_1 - C_{t_1}$$

Again, since V_{t_2} must be proportional to the gate opening multiplied by the square root of the head, then, for uniform gate motion:

$$V_{t_2} = \left(1 - \frac{t_2}{T}\right) B_0 \sqrt{H_0 + h_{t_2}} \quad \text{.....(9)}$$

Substituting this value of V_{t_2} in the equation for h_{t_2} the latter becomes

$$h_{t_2} = \frac{a}{g} V_1 - \frac{a}{g}\left(1 - \frac{t_2}{T}\right) B_0 \sqrt{H_0 + h_{t_2}} + h_1 - C_{t_1}.$$

Substituting the symbol, R_{t_1}, in place of $\frac{a}{g} V_1 + h_1 - C_{t_1}$, and the symbol, $\sqrt{S_{t_2}}$, for $\frac{a}{g}\left(1 - \frac{t_2}{T}\right) B_0$,

$$h_{t_2} = R_{t_1} - \sqrt{S_{t_2}(H_0 + h_{t_2})}$$

Squaring: $(h_{t_2} - R_{t_1})^2 = S_{t_2}(H_0 + h_{t_2})$

Expanding: $h_{t_2}^2 - 2 h_{t_2} R_{t_1} + R_{t_1}^2 = S_{t_2} H_0 + S_{t_2} h_{t_2}$

Collecting: $h_{t_2}^2 - h_{t_2}(S_{t_2} + 2 R_{t_1}) + R_{t_1}^2 - S_{t_2} H_0 = 0$

Solving: $h_{t_2} = \frac{1}{2}\left\{(S_{t_2} + 2 R_{t_1}) \pm \sqrt{(S_{t_2} + 2 R_{t_1})^2 - 4 R_{t_1}^2 + 4 S_{t_2} H_0}\right\}$

Simplifying: $h_{t_2} = \frac{1}{2}\left\{(S_{t_2} + 2 R_{t_1}) \pm \sqrt{S_{t_2}(S_{t_2} + 4 R_{t_1} + 4 H_0)}\right\}$..(10)

In a similar manner, the value of h, at any time in any interval, may be found.

Second: Derivation of Formulas with Friction Included.—In the foregoing analysis no account has been taken of the effect of frictional losses of head in the penstock. As the velocity in the penstock is gradually destroyed, the friction head is gradually recovered and is

added to the net head, producing discharge through the turbine gates. In any penstock the total frictional losses may be assumed to be proportional to the square of the velocity of flow, and may be represented by a coefficient, F, such that $FV^2 =$ total loss in feet of head for a velocity, V. The velocity head recovered at any time in the first interval, therefore, is $F\,V_0^2 - F\,V_{t_1}^2$. This head must then be added to the sum of $H_0 + h_{t_1}$, in determining the relation between velocity, gate opening, and head, as given by Equation (6).

Thus: $$V_{t_1} = \left(1 - \frac{t_1}{T}\right) B_0 \sqrt{H_0 + h_{t_1} + F\,V_0^2 - F\,V_{t_1}^2}$$

Squaring: $$V_t^2 = \left(1 - \frac{t_1}{T}\right)^2 B_0^2\,(H_0 + h_{t_1} + F\,V_0^2 - F\,V_{t_1}^2)$$

Collecting: $$V_{t_1}^2 \left\{1 + \left(1 - \frac{t_1}{T}\right)^2 B_0^2\,F\right\} = \left(1 - \frac{t_1}{T}\right)^2 B_0^2\,(H_0 + h_{t_1} + F\,V_0^2)$$

Solving, and multiplying numerator and denominator by $\frac{a}{g}$:

$$V_{t_1} = \sqrt{\frac{\left(\frac{a}{g}\right)^2 \left(1 - \frac{t_1}{T}\right)^2 B_0^2\,(H_0 + h_{t_1} + F\,V_0^2)}{\left(\frac{a}{g}\right)^2 + \left(\frac{a}{g}\right)^2 \left(1 - \frac{t_1}{T}\right)^2 B_0^2\,F}}$$

Substituting S_{t_1} for $\left(\frac{a}{g}\right)^2 \left(1 - \frac{t_1}{T}\right)^2 B_0^2$,

$$V_{t_1} = \sqrt{\frac{S_{t_1}\,(H_0 + h_{t_1} + F\,V_0^2)}{\left(\frac{a}{g}\right)^2 + S_{t_1}\,F}} \qquad \ldots\ldots\ldots\ldots\ldots (11)$$

Inserting this value of V_{t_1} in Equation (5) for h_{t_1},

Then: $$h_{t_1} = \frac{a}{g}\,V_0 - \frac{a}{g}\sqrt{\frac{S_{t_1}\,(H_0 + h_{t_1} + F\,V_0^2)}{\left(\frac{a}{g}\right)^2 + S_{t_1}\,F}}$$

Solving for h_{t_1} in the same manner as that used in obtaining Equation (7), and substituting R_0 in place of $\frac{a}{g}\,V_0$, and Z_1 in place of

$$\frac{\left(\frac{a}{g}\right)^2 S_{t_1}}{\left(\frac{a}{g}\right)^2 + S_{t_1}\,F},$$

Then: $h_{t_1} = \frac{1}{2}\left\{(2R_0 + Z_1) \pm \sqrt{Z_1^2 + 4Z_1(R_0 + H_0 + F V_0^2)}\right\}$ ..(12)

In a similar manner, the equation for h_{t_2} is obtained by adding $F V_0^2 - F V_{t_2}^2$ under the root sign in Equation (9) and inserting the resulting value of V_{t_2} in Equation (8). Without performing the operations, which are similar to the above, the result may be written down at once.

$$h_{t_2} = \frac{1}{2}\left\{(2R_{t_1} + Z_2) \pm \sqrt{Z_2^2 + 4Z_2(R_{t_1} + H_0 + F V_0^2)}\right\} \quad ..(13)$$

Where: $R_{t_1} = \left(\frac{a}{g} V_1 + h_1 - C_{t_1}\right)$

$$C_{t_1} = \frac{2a}{g}(V_0 - V_{t_1})$$

$$Z_2 = \frac{\left(\frac{a}{g}\right)^2 S_{t_2}}{\left(\frac{a}{g}\right)^2 + S_{t_2} F}$$

and $$S_{t_2} = \left(\frac{a}{g}\right)^2 \left(1 - \frac{t_2}{T}\right)^2 B_0^2$$

Similarly, the value of h, at any time in any interval, may be found.

When the gate motion is not uniform, the value of S_t has to be made equal to $x^2 \left(\frac{a}{g}\right)^2 B_0^2 \left(1 - \frac{t}{T}\right)^2$, where x is a constant or variable coefficient, which may be determined by plotting the curve of gate opening on a time base. For uniform motion, the curve would be a straight line, the ordinate of which, at any time, t, is $\left(1 - \frac{t}{T}\right) B_0$. For any other than uniform motion, the ordinates of the straight line would be multiplied by the constant or variable, x, which, if the motion varied in a regular manner, might sometimes be expressed in terms of t, either graphically or analytically, from the known relation between the governor movements and the gate opening. If the motion were not regular, a graphical solution only could be obtained.

Tables 4 and 5 show the values of h_t obtained by using the formulas in the examples worked out by arithmetic integration in Tables 1, 2, and 3.

It should be noted here, by way of caution, that, in calculating the value of h, the two terms inside the larger brackets of the foregoing formulas are frequently so large and their difference so small as to make it necessary to perform the work by logarithms. The correct result cannot be obtained in such cases by using an ordinary 10-in. slide-rule.

TABLE 4.—By Formulas.

Data: $L = 820$ ft. $V_0 = 11.75$ ft. per sec. $H_0 = 165$ ft. $T = 2.1$ sec. $a = 4\,680$ ft. per sec. Friction neglected.

(1)	(2)	(3)	(4)
Interval.	Gate, B.	Rise of pressure, h_t.	Velocity, V.
0	0.9148	0.0	11.7500
¼	0.8766	12.12	11.6664
½	0.8385	25.53	11.5740
¾	0.8004	40.41	11.4713
1	0.7623	56.96	11.3571
1¼	0.7242	69.94	11.1004
1½	0.6861	83.72	10.8205
1¾	0.6480	98.31	10.5149
2	0.6099	113.64	10.1805
2¼	0.5718	125.78	9.7505
2½	0.5337	138.05	9.2909
2¾	0.4956	150.32	8.8005
3	0.4574	162.42	8.2766
3¼	0.4193	171.97	7.6970
3½	0.3811	181.10	7.0899
3¾	0.3430	189.43	6.4574
4	0.3049	196.77	5.7994
4¼	0.2668	202.49	5.1146
4½	0.2287	207.14	4.4118
4¾	0.1906	211.00	3.6959
5	0.1525	213.79	2.9680
5¼	0.1143	215.75	2.2303
5½	0.0762	216.71	1.4887
5¾	0.0381	216.95	0.7446
6	0.0	216.57	0.0

It is now interesting and instructive to compare the results given by the foregoing formulas with those given by other formulas which have been proposed to determine the rise of pressure caused by gradually stopping the flow of water in pipes. For this purpose, Fig. 3 has been prepared. There are three such fundamentally different formulas known to the writer, namely, those of Mr. L. Allievi,* M. M. Warren,†

* The writer was unable to obtain an English translation of Mr. Allievi's paper, but his formula is commonly given as follows:

$$h = \frac{N H}{2} + H\sqrt{\frac{N^2}{4} + N}$$

Where h = rise of pressure, in feet, above normal;
H = normal net head, in feet;

$$N = \left(\frac{L V}{g T H}\right)^2;$$

L = length of penstock, in feet;
V = velocity of water in penstock, in feet per second;
T = duration of gate closure, in seconds;
g = acceleration due to gravity, in feet per second per second.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, pp. 238, 242. Mr. Warren's formula is:

$h = \frac{L V}{g\left(T - \frac{L}{a}\right)}$, where a = velocity of the pressure wave, and the other symbols have the same significance as in this paper.

TABLE 5.—By Formulas.

Data: $L = 6\,337$ ft. $V_0 = 15.055$ ft. per sec. $H_0 = 1\,260$ ft. $T = 69.5$ sec. $a = 3\,647$ ft. per sec. Friction head, $h_f = 81$ ft.

Non-uniform Gate Motion.

(1)	(2)	(3)	(4)
Interval.	Gate, B.	Rise of pressure.	Velocity.
0	0.4241	0.	15.0550
1	0.4168	18.68	14.9040
2	0.4084	20.42	14.6137
3	0.3995	25.04	14.3210
4	0.3895	30.74	13.9931
5	0.3784	36.52	13.6351
6	0.3657	44.12	13.2063
7	0.3513	52.07	12.7250
8	0.3354	59.96	12.1854
9	0.3180	68.27	11.5898
10	0.2991	76.53	10.9388
11	0.2781	87.85	10.2097
12	0.2544	99.30	9.3792
13	0.2285	110.13	8.4551
14	0.2009	118.88	7.4606
15	0.1719	128.15	6.4024
16	0.1411	136.68	5.2746
17	0.1083	146.75	4.0599
18	0.0725	156.94	2.7301
19	0.0363	156.13	1.3638
20	0.0	159.21	0.

Assoc. M. Am. Soc. C. E., and H. C. Vensano,* M. Am. Soc. C. E. The formula by Allievi gives the maximum excess pressure caused by stopping the flow of a column of water, assumed as an incompressible fluid, moving in an inextensible pipe, and neglecting the effect of friction. Mr. R. D. Johnson,† Mr. A. H. Gibson,‡ and W. F.

* *Transactions*, Am. Soc. C. E., Vol. LXXIX, pp. 289–299; and Vol. LXXXII, p. 185. Mr. Vensano's formula is:

$h = \frac{2\,l\,V}{g\,T}$, with the limitation that h can never be greater than $\frac{V\,a}{g}$.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, pp. 277–281. Mr. Johnson's formula is:

$$H_{\text{max.}} = \frac{2\,M\,Y}{N^2}\left(M + \sqrt{M^2 + N^2}\right)$$

where $H_{\text{max.}}$ = maximum rise of pressure, in feet, above normal; $M = L\,V$; and $N = 2\,g\,Y\,T$, in which Y is the normal net head in feet, and the other symbols have the same significance as in this paper.

‡ "Water Hammer in Hydraulic Pipe Lines", by A. H. Gibson. Mr. Gibson's formula is: (In order to prevent confusion with the foregoing, some changes have been made in the nomenclature):

$$p' = \frac{w}{g}\left[\left(\frac{L}{A}\cdot\frac{A_1}{T}\right)^2 + \frac{L}{A}\cdot\frac{A_1}{T}\sqrt{2\,g\,H + \left(\frac{L}{A}\cdot\frac{A_1}{T}\right)^2}\right]$$

where p' = rise in pressure, in pounds per square foot, behind the valve at the instant when closure is complete, and therefore when p is maximum;

A = cross-sectional area of penstock, in square feet;

A_1 = maximum effective area of valve opening, in square feet;

w = weight of a cubic foot of water;

and the other symbols have the same significance as in this paper.

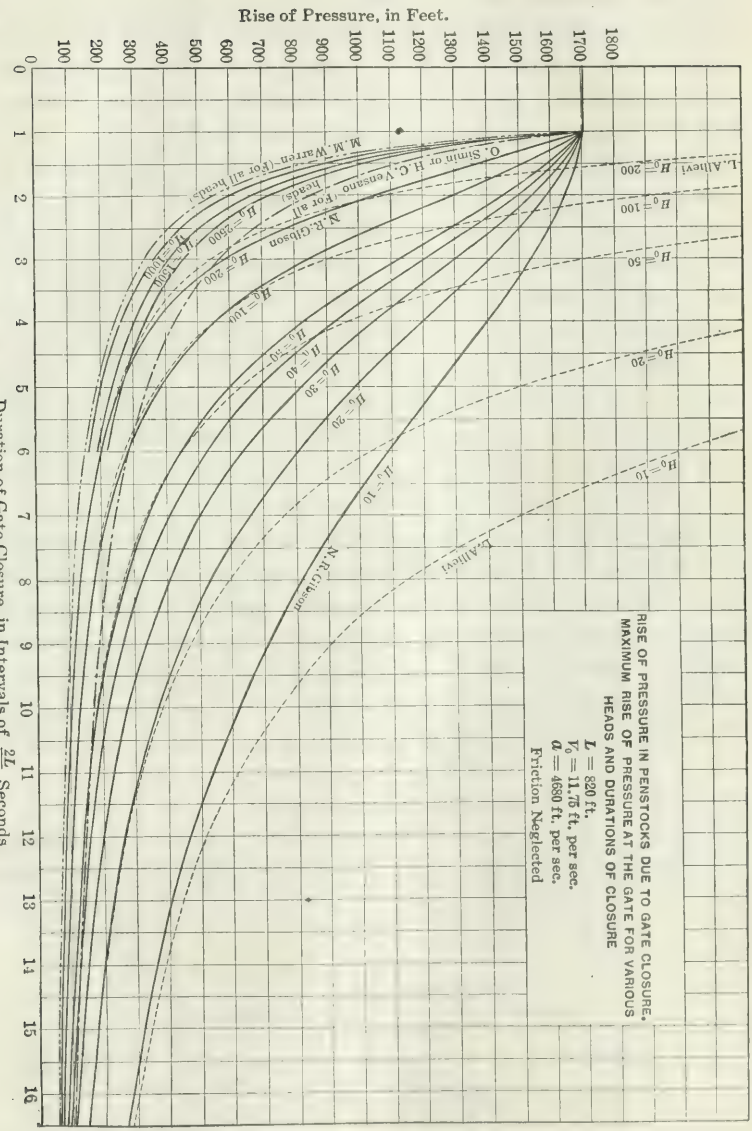
Rise of Pressure, in Feet.
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Duration of Gate Closure, in Intervals of $\frac{2L}{a}$ Seconds
RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE.
MAXIMUM RISE OF PRESSURE AT THE GATE FOR VARIOUS HEADS AND DURATIONS OF CLOSURE
$L = 820$ ft.
$V_0 = 11.76$ ft. per sec.
$a = 4650$ ft. per sec.
Friction Neglected
L. Allievi
$H_0 = 200$
$H_0 = 100$
$H_0 = 50$
$H_0 = 20$
$H_0 = 10$
L. Allievi
N. R. Gibson
$H_0 = 10$
$H_0 = 20$
$H_0 = 30$
$H_0 = 40$
$H_0 = 50$
$H_0 = 100$
$H_0 = 200$
M. R. Gibson — For all heads
$H_0 = 2000$
$H_0 = 1500$
$H_0 = 1000$
O. Simin or H. C. Vensano — For all heads
M. M. Watson — For all heads

Fig. 3.

Uhl,* M. Am. Soc. C. E., have each presented a similar formula, with different nomenclature and somewhat differently arranged, but all give the same result as Allievi's formula. Mr. Johnson has also presented a method of determining the pressure-time curve resulting from the same assumption,† and Mr. A. H. Gibson has shown how friction may be taken into account by successive calculations.‡

The limitations of the Allievi formula are well known to these authors, as they specifically refer to them; they are known, also, no doubt, to many others who use that formula, because it is readily seen that, for zero duration of closure, the formula gives an infinite pressure rise, and this cannot be so because the finite value of maximum water-hammer, for instantaneous closure, has been proved by Professor Joukovsky. Allievi's formula takes into account the effect of the net head on the phenomena that occur during closure. In Fig. 3, there are shown in dotted lines a series of Allievi curves for various heads, and, for the data assumed, these show the maximum rises of pressure as ordinates and durations of closure as abscissas. For convenience, the time abscissas have been marked off in intervals, instead of seconds. The curves, it will be noted, become very steep as the duration of closure becomes short, and, in the limit, appear to approach infinity as the duration of closure approaches zero. For low heads Joukovsky's limit of maximum water-hammer is passed by these curves at points of relatively long duration of gate closure. In Fig. 3, a similar series

* *Transactions*, Am. Soc. Mech. Engrs., Paper No. 1354. Mr. Uhl's formula is:

$$\frac{D\,H}{H} = \frac{n}{2}\left(n + \sqrt{n^2 + 4}\right)$$

where DH = rise of pressure, in feet, above normal;

$n = \dfrac{L\,V}{g\,T\,H}$, in which the symbols have the same significance as in this paper.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, pp 277–281. Mr. Johnson's formulas, to be solved as indicated in his discussion, are:

$$t = T\left\{\frac{\sqrt[n]{x} - 1}{\sqrt[n]{x}}\right\}$$

where $x = R\left\{\dfrac{H + \sqrt{y_0}}{J - \sqrt{y_0}}\right\}$, R being $\dfrac{J - \sqrt{Y}}{H + \sqrt{Y}}$,

and $n = \sqrt{\left(\dfrac{2\,g\,T\,Y}{L\,V}\right)^2 + 1}$, $H = \dfrac{L\,V}{2\,g\,T\sqrt{Y}}(n - 1)$,

$$J = \frac{L\,V}{2\,g\,T\sqrt{Y}}(n + 1)$$

‡ "Water Hammer in Hydraulic Pipe Lines", by A. H. Gibson.

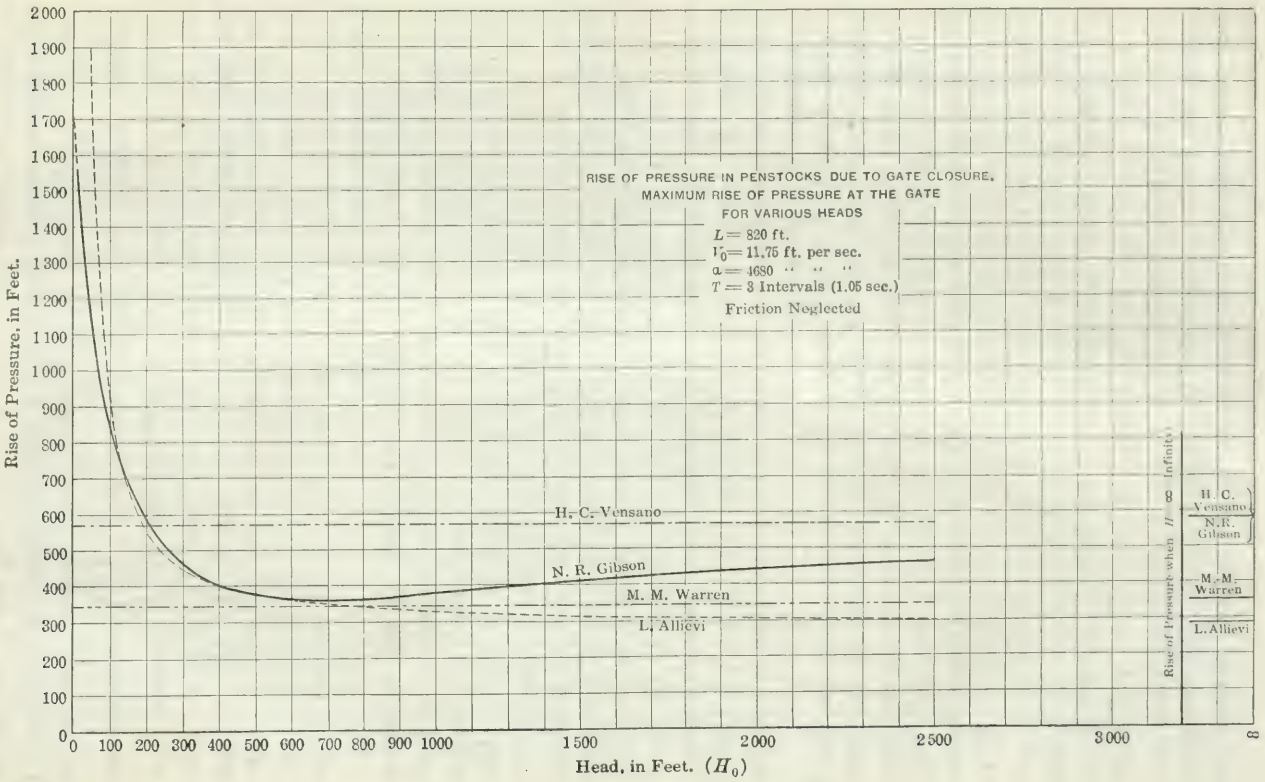
RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE.
MAXIMUM RISE OF PRESSURE AT THE GATE
FOR VARIOUS HEADS
L = 820 ft.
V_0 = 11.75 ft. per sec.
a = 4680 " " "
T = 3 Intervals (1.05 sec.)
Friction Neglected
H. C. Vensano
N. R. Gibson
M. M. Warren
L. Allievi
Rise of Pressure, in Feet.
Head, in Feet. (H_0)
Rise of Pressure when H = Infinity
H. C. Vensano
N. R. Gibson
M. M. Warren
L. Allievi
0 100 200 300 400 500 600 700 800 900 1000 1500 2000 2500 3000 ∞

FIG. 4.

of curves for various heads has been plotted from the result obtained by using the writer's formulas, and these are shown in full lines. It will be noted that these curves show maximum water-hammer for any duration of closure from zero up to one interval $\left(\frac{2\,L}{a}\right)$, and, as the duration of closure becomes longer, the resulting rise of pressure finally approaches, within certain limits, the value shown by the Allievi curve. For low heads and short durations of closure, the Allievi curves generally show higher values of pressure rise than those of the writer; but, for high heads and short durations of closure, they show lower values of pressure rise. This interesting fact will be referred to later when discussing the pressure-time curves.

The formulas of Messrs. Warren and Vensano do not include the net head as a factor, and, therefore, each may be represented, as shown in Fig. 3, by a single curve for all heads. Although these two formulas are objectionable, because they neglect the influence of the net head, it is evident, from the position of the curves representing them in Fig. 3, that under certain special conditions they will give, at least approximately, correct results. These conditions will be referred to later when discussing the pressure-time curves, but it is interesting to note here that Mr. Warren's formula gives approximately the correct results when the net head is such that the excess pressure will be a minimum for any given duration of closure. Mr. Vensano's formula gives approximately the correct result when the net head is very high with respect to the velocity destroyed.

Fig. 4 has been drawn to show the manner in which the rise of pressure varies with the net head. The results shown by the curves in Fig. 3 are plotted for a given duration of closure, ordinates representing excess pressure, but abscissas representing net head (H_0). It is thus seen that, for a given duration of closure, the excess pressure becomes less as the head increases, until at a certain value of H_0 the minimum rise of pressure is reached, and, as H_0 continues to increase, the rise of pressure also increases until it reaches a final value, when H_0 is infinite. In the example shown in Fig. 4, L is only 820 ft., and this length would represent the limit of gravity head. The curves, however, have been extended for higher heads, on the assumption, of course, that the head is produced by applied pressure. The corresponding curves obtained from the results of the Allievi, Warren, and Vensano formulas are also clearly shown in Fig. 4.

Pressure-Time Curves.

The pressure-time curves, Figs. 5, 6, and 7, show clearly the changes in pressure that take place during the closure of the gates under various typical conditions. The corresponding curves obtained from the

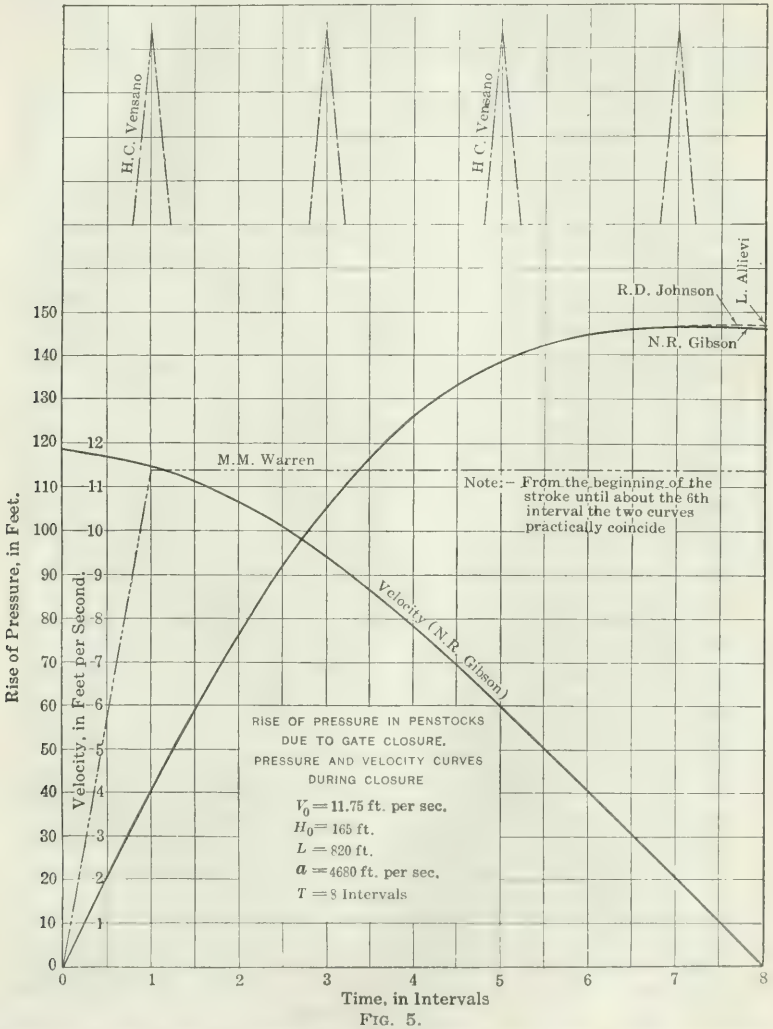
H.C. Vensano
H.C. Vensano
R.D. Johnson
L. Allievi
N.R. Gibson
M.M. Warren
Note:— From the beginning of the stroke until about the 6th interval the two curves practically coincide
Velocity (N.R. Gibson)
RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE. PRESSURE AND VELOCITY CURVES DURING CLOSURE
V_0 = 11.75 ft. per sec.
H_0 = 165 ft.
L = 820 ft.
a = 4680 ft. per sec.
T = 8 Intervals
Rise of Pressure, in Feet.
Velocity, in Feet per Second.
Time, in Intervals
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150
0 1 2 3 4 5 6 7 8
1 2 3 4 5 6 7 8 9 10 11 12

FIG. 5.

formulas of Allievi, Warren, and Vensano have been shown in these figures for the purpose of comparison with those of the writer, and in order that the various results may be studied and explained. In some

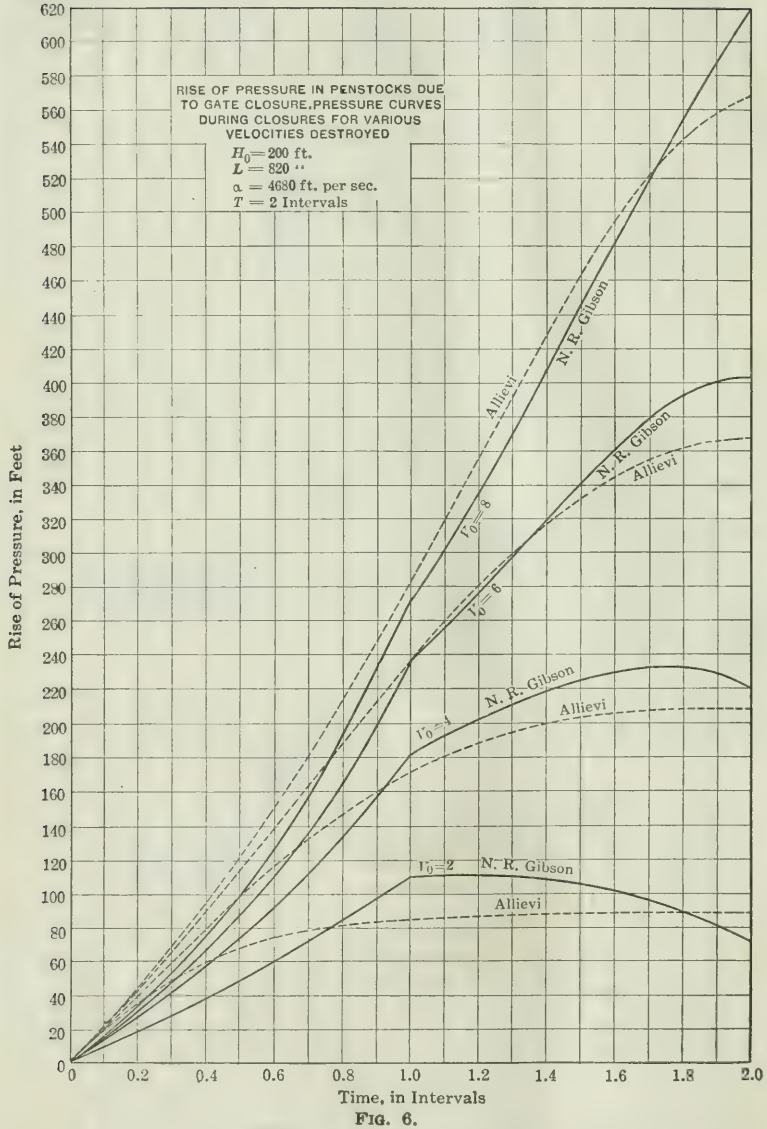
RISE OF PRESSURE IN PENSTOCKS DUE TO GATE CLOSURE. PRESSURE CURVES DURING CLOSURES FOR VARIOUS VELOCITIES DESTROYED
H_0 = 200 ft.
L = 820 "
a = 4680 ft. per sec.
T = 2 Intervals
Rise of Pressure, in Feet
Time, in Intervals
N. R. Gibson
Allievi
V_0=8
V_0=6
V_0=4
V_0=2
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500 520 540 560 580 600 620
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0

FIG. 6.

cases velocity-time curves have also been drawn, in order to show the variable rate of retardation caused by uniform gate closure.

It will be noted first that Mr. R. D. Johnson's pressure-time curve, based on Allievi's formula, or indeed Allievi's formula itself, gives the maximum pressure rise always at the end of the closing time. Mr. Warren's curve shows the excess pressure rising uniformly to a maximum at the end of the first interval and then remaining constant until

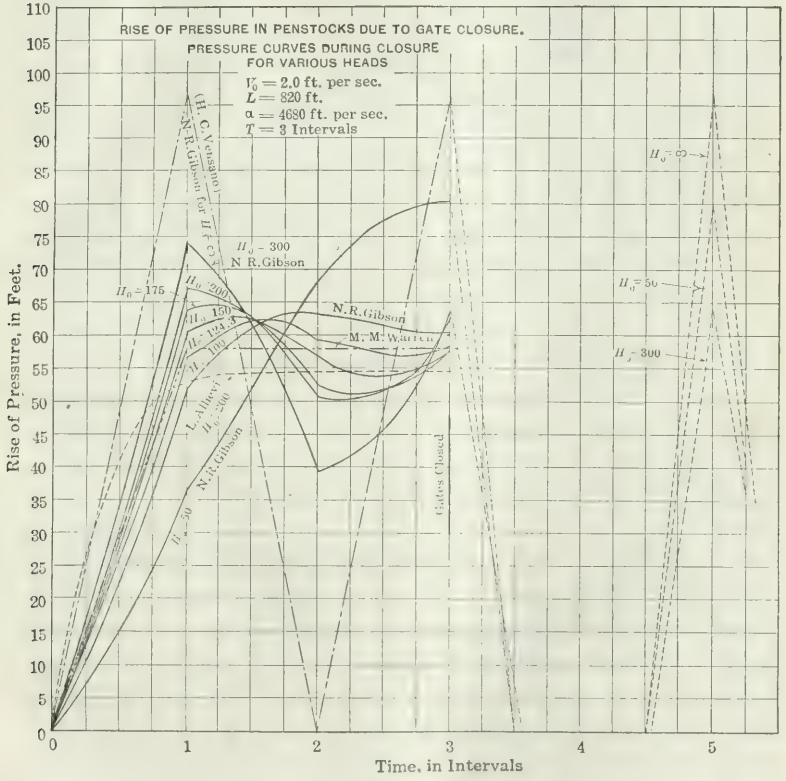

FIG. 7.

the end of the closing time. Mr. Vensano's curve shows the excess pressure rising uniformly to a maximum at the end of the first interval, falling uniformly to zero at the end of the second interval, and repeating this vibration until the end of the closing time. The writer's pressure-time curves may be similar to any one of these three, depending on the duration of closure and the net head acting on the orifice, or on both of these factors.

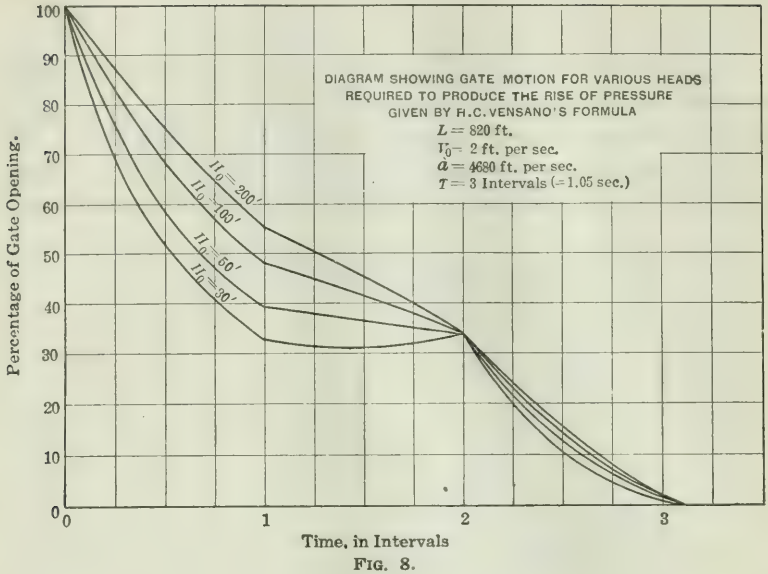
DIAGRAM SHOWING GATE MOTION FOR VARIOUS HEADS
REQUIRED TO PRODUCE THE RISE OF PRESSURE
GIVEN BY H.C. VENSANO'S FORMULA
L = 820 ft.
V0 = 2 ft. per sec.
a = 4680 ft. per sec.
T = 3 Intervals (= 1.05 sec.)
H0 = 200′
H0 = 100′
H0 = 50′
H0 = 30′
Percentage of Gate Opening.
Time, in Intervals
0
10
20
30
40
50
60
70
80
90
100
1
2
3

FIG. 8.

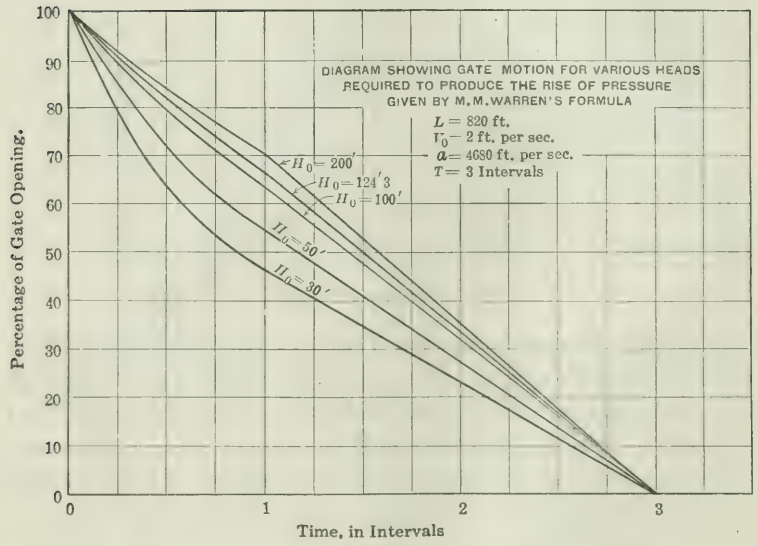
DIAGRAM SHOWING GATE MOTION FOR VARIOUS HEADS
REQUIRED TO PRODUCE THE RISE OF PRESSURE
GIVEN BY M.M. WARREN'S FORMULA
L = 820 ft.
V0 = 2 ft. per sec.
a = 4680 ft. per sec.
T = 3 Intervals
H0 = 200′
H0 = 124′3
H0 = 100′
H0 = 50′
H0 = 30′
Percentage of Gate Opening.
Time, in Intervals
0
10
20
30
40
50
60
70
80
90
100
1
2
3

FIG. 9.

Commencing with Fig. 5, it is seen that the pressure-time curve, under the conditions stated, is almost identical with that given by Mr. R. D. Johnson, as far as can be detected by the eye. A careful consideration of the writer's formula, however, will reveal the fact that the pressure-time curve obtained from it is not continuous, but is made up of a series of curves, each being one interval long, and cusps or changes of curvature occur at the end of each interval. Under other conditions, the cusps are plainly visible, as, for example, in Figs. 6 and 7. For any finite conditions, these cusps, theoretically, do not disappear, that is to say, the tangent at the end of one interval never exactly equals the tangent at the beginning of the next interval. Furthermore, the slope at the beginning of the writer's curve is never exactly the same as the slope of the Johnson curve. For all practical purposes, however, the pressure-time curves shown in Fig. 5 (and numerous other curves for similar conditions) are almost identical, and the maximum pressure rise occurs in both of them at the end of the stroke. This is interesting because the two theories from which the respective formulas are derived are totally different. When the conditions are such as to produce practically identical pressure-time curves similar to the foregoing, it indicates that the effect of the compressibility of water, the elasticity of the pipe walls, and the resulting harmonic vibrations in the water column may be neglected, and in such cases the assumptions, on which the Allievi formula has been based, are practically sufficient.

The results obtained from the Warren and Vensano curves are much at variance with the foregoing, as indicated in Fig. 5, and the writer is of the opinion that neither of these formulas can apply for all values of H_0, and, only under certain conditions, which will be mentioned later, are they approximately correct.

As shown in Fig. 3, the maximum pressure rise given by the writer's formulas, as would be expected, is nearly always less than that given by the Allievi formula when the duration of closure is short and when $\frac{H_0}{V}$, the ratio of head to velocity destroyed, is small. When the ratio of head to velocity destroyed is large, the reverse is the case, and, when H_0 becomes infinitely large, the writer's formulas agree with that of Mr. Vensano and give results twice as great as Allievi's formula for any finite duration of closure greater than one interval. Even

when the values of maximum pressure rise, as given by the Allievi formula, are in close agreement with those of the writer, both for short and long durations of closure, the respective shapes of the pressure-time curves may be different. Figs. 6 and 7 show clearly the characteristics of the pressure-time curves in a number of typical cases, and a close study of them, together with Fig. 4, will make clear the relation that exists between the rise of pressure and the net head for any velocity destroyed. The shapes of the pressure waves that continue after the gates have been closed, until damped out by friction, are also indicated in Figs. 6 and 7.

Referring now to Fig. 7, it will be noted that the particular case when Mr. Warren's formula gives approximately correct results is when the head is such that the rise of pressure due to gate closure is a minimum for the particular velocity destroyed. This occurs when $H = \frac{3}{7}\frac{aV}{g}$.

Fig. 7 shows, also, the shapes of the pressure-time curves for various heads, and indicates the variation in time when the maximum pressure occurs during closure.

The foregoing examples, except Tables 2 and 5, have all been calculated for uniform gate motion. As the writer's formulas may be applied for any variable rate of gate motion, it is interesting to determine the nature of the gate motion for various heads that would be required to produce diagrams similar to those proposed by Messrs. Warren and Vensano. A diagram of the shape proposed by Mr. Warren may be produced by the simple expedient of increasing the speed of gate travel at the end of the first interval to double the rate of its motion during the first interval and maintaining this double rate until the end of the stroke. Such an operation, though giving the same shape to the pressure-time diagram, will not give, of course, the same rise of pressure as for the same duration of uniform closure, but the latter is given by a gate motion similar to that shown in Fig. 9. If a pressure-time diagram of the same shape as Mr. Vensano's is desired for a low head, a very complicated gate motion would be required, such, for example, as that shown in Fig. 8. In a similar manner, the gate motion required to produce any suggested form of pressure-time curve may be determined.

The manner in which excess pressures vary from gate to origin has been explained in Miss Simin's translation of Professor Joukovsky's paper, although therein worked out on the erroneous assumption of uniform retardation of the velocity of the water flowing in the pipe.

O. V. Kruse, Assoc. M. Am. Soc. C. E., who was associated with the writer at the time this paper was first written, has made an original study of the variation in excess pressure from gate to origin, based on the writer's application of Joukovsky's theory to slow-closing gates. The results of this study, which will appear as a discussion, together with other comments from him, prove that when the duration of uniform closure is less than $\frac{2\,L}{a}$, the maximum pressure is exerted along the pipe to a point where the distance to the origin is equal to $\frac{T\,a}{2}$. From that point to the origin the pressure reduces uniformly to zero. When the duration of uniform closure is equal to, or greater than, $\frac{2\,L}{a}$, the maximum rise of pressure occurs at the gate, and from there to the forebay or origin reduces to zero, uniformly along the length of the pipe.

Fall in Pressure.

Professor Joukovsky's theory of pressure waves applies also to the fall in pressure produced by opening a valve or gate at the end of a pipe line, and the application of his theory to the case when a valve is gradually opened may be made in a similar manner to the foregoing. It is obvious, of course, that the fall in pressure caused by opening a valve in a certain time is not precisely the same as the rise in pressure caused by closing the valve in the same time, because the rate of acceleration cannot be the same as the rate of retardation. It would lengthen this paper unduly, however, to do more than state here that the formulas for fall in pressure may be obtained in a similar manner to those given herein, the principles being the same in both cases.

Experimental Results.

The writer has not yet had an opportunity of testing the formulas presented in this paper by experiments, to show, not only the correctness of the values of maximum pressure rise, but also the shapes of the pressure-time curves under various conditions. It is expected, however,

that an opportunity of doing so may possibly occur in the not too distant future. In the meantime this paper is presented in the hope that it will receive criticism or confirmation from those interested in the subject.

The experimental results obtained by Mr. Vensano, and presented in his paper entitled "Pulsations in Pipe Lines", give some striking confirmation of the correctness of the writer's formulas, not only as regards the maximum pressure rise, but also as indicating the resemblance between the calculated and observed pressure-time curves, both before and after the gate was completely closed, and at various points along the pipe line, as well as at the gate.* These, however, were carried out under a very high head (1 260 ft.), and could hardly be accepted as a general proof.

The writer also proposes to show how the pressure-time diagram may be used to determine the velocity of flow in a pipe, for the purpose of measuring the rate of discharge previous to the closing of the gate. This will be made the subject of another paper to be written at a future time.

In conclusion, the writer desires to accord credit to Mr. O. V. Kruse for the valuable help he has given in the preparation of this paper, to Mr. R. L. Hearn for the work of preparing Figs. 8 and 9, and to Mr. R. D. Johnson for the many helpful suggestions he has kindly made.

* See discussion by Norman R. Gibson in *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 236.

DISCUSSION

Mr. Kruse.

OTTO V. KRUSE,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has studied the phenomena of water-hammer, and offers a short analysis of Miss O. Simin's translation of Prof. Joukovsky's notable work, and an elaboration of certain parts of Mr. Gibson's paper.

Any pipe under pressure and containing water in motion may be assumed as a unit of energy under a condition of equilibrium. If we wish to stop the column of water, it means the application of a certain amount of force during a certain time to bring about the conversion of energy and restore the equilibrium. In analyzing the phenomena which take place, the writer believes that the most important point to study is the rate of destruction of velocity as affected by the net head, because this determines to a large extent the rate of pressure rise and hence the maximum pressure rise. Apparently, Allievi used this as a basis for the development of his formula, and this, of course, applies to Johnson's formula, which is the same as Allievi's. Any formulas, such as Warren's and Vensano's, which do not take this into account must be incorrect, except for one set of conditions. The curves given by Mr. Gibson show how far from the truth they may be.

In accordance with the ordinary methods of integration, it may be considered that the gate movement throughout the duration of the stroke is made up of an infinite number of small instantaneous movements. In the case of the formulas of Allievi and Johnson, which involve the dynamic forces, each little movement of the gate produces a pressure:

$$h = \frac{L\,dv}{g\,dt}$$

The conditions existing, then, at the end of any small movement are:

1. The original velocity has been decreased by the amount of the velocity destroyed during this first movement;
2. A dynamic pressure has been created, due to the destruction of velocity.

A new velocity now exists, which is dependent, not on the original net head, but on a new head made up of the original net head and the dynamic head. As brought out by Mr. Johnson, these curves of dynamic pressure rise and decrease in velocity may be calculated by a simple method of arithmetic integration, using only the relation, $h = \frac{L\,dv}{g\,dt}$. The Allievi or Johnson formula will give a figure for maximum pressure rise, and Mr. Johnson has also developed a formula for the shape

* Philadelphia, Pa.

Mr. Kruse.

of the pressure curve, these formulas giving the same results as can be obtained approximately by arithmetic integration.

The question to be considered now is: To what extent, and how, do the compressibility of water and extension of the pipe walls affect the pressure and velocity curves? We know that, as the time of closing grows shorter, the Allievi formula approaches infinity. On the other hand, Joukovsky proved that the maximum pressure which obtained for instantaneous closing depended on the velocity destroyed and the speed of propagation of the pressure wave, and could not exceed "maximum water-hammer." It is not difficult to see that, for a long closing time, the compressibility of the water and extension of the pipe walls have very little effect on the maximum pressure, but, as the time of closing grows shorter, the properties of the materials have a greater and greater effect. The writer believes that Mr. Gibson's formulas have successfully supplied this missing gap between Allievi's and Joukovsky's theories, and serve to give correct results for all conditions.

Professor Joukovsky made a great many experiments to determine the magnitude of maximum water-hammer and the speed of propagation of the pressure wave, all based on instantaneous closing of the gates. The details of the experiments will not be given here, but the following is a synopsis of the theory, taken verbatim from Miss Simin's translation:

"In Fig. [10] let $A\ B$ be a pipe, in which water flows with velocity, v, from the origin A, past the gate, B. If, now, the flow is suddenly stopped by a rapid shutting of the gate, B, the kinetic energy of the water column, $A\ B$, will cause an increase of pressure in the pipe.

"Let us consider the column of water, $A\ B$, as divided into n very small equal sections, 1, 2, 3,....$(n - 1)$ and n.

"The phenomena of water-hammer take place in a series of cycles, each consisting of four processes, as follows:

"(1) Section 1, meeting, in the gate, an obstacle to its movement, will be compressed and will stretch the pipe wall surrounding it. All the kinetic energy of this section of water will be used up (a) in its own compression, resulting in the increase of pressure by an increment, P, and (b) in the corresponding stretching of the walls in section 1 of the pipe. As a result of this action, section 1 of the water column has left vacant behind itself a small space, to be occupied by a part of the next arriving section 2. Consequently it is only after section 1 has been stopped and compressed, and after the small space thus left has been filled, that section 2 can be arrested and compressed.

"Now the kinetic energy of section 2 must be expended in some way. Will it increase the pressure upon the gate, which has already been caused by the arrest of section 1? No, and for the following reason:

"The pressure upon the gate depends entirely upon the pressure, P, sustained by section 1, which is now in static condition.

"The pressure upon the gate could therefore be increased only if section 1 could be further compressed, and this could take place only

if the pressure upon the surface between it and section 2 (which we may imagine to be a thin piston) could be greater from the side of section 2 than it is from the side of section 1; and this is impossible, because section 2 has only the same kinetic energy as section 1, and this energy will (as in the case of section 1) be used up entirely in compressing the water of the section (section 2) only to the same additional pressure, P, and in stretching that part of the walls surrounding section 2. Mr. Kruse.

"The same is true of each following section, 3, 4,.... $(n - 1)$ and n; each of these sections, as it is arrested, being compressed to the pressure, P.

"During process (1) a small quantity of water flows from the reservoir into the pipe, to occupy the space formed by the compression of the water and the extension of the pipe walls.

"Finally, when all the sections have been arrested, the entire column will be under the pressure, P. The entire energy of the water column is now stored (as potential energy) in elastic deformation, viz., in the compression of the water column and in the extension of the pipe walls.

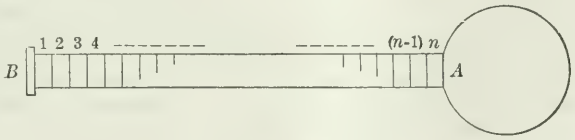

FIG. 10.

"But this condition cannot be maintained; for

"(2) As soon as the additional pressure, P, has been produced in the last section, n, the water in that section will again expand, and the walls of that section of the pipe will again contract, restoring the original conditions in that section, and pushing the water of that section back into the reservoir from which the pipe issues, and restoring the original normal pressure in section n.

"This operation will now be repeated by each section $(n - 1)$,....4, 3, etc., in turn, until all the potential energy, stored in the water column when it was under the pressure, P (neglecting the portion lost in friction), has been reconverted into kinetic energy.

"During process (2) the water which entered the pipe during process (1) is forced back into the reservoir.

"The condition of the water column is now what it was just before the gate was closed, except that its velocity, v, has now the opposite direction, *i. e.*, toward the origin.

"(3) The kinetic energy of the water column, moving toward the origin or away from the gate, is now reconverted into potential energy, which manifests itself in an extension of volume of the water to a subnormal pressure beginning with section 1, and concluding only when the entire water column has been reduced to the subnormal pressure.

"During process (3) water continues flowing from the pipe into the reservoir.

"(4) When the subnormal pressure has been established throughout the length of the pipe, and all the water has come to rest, the water from the reservoir will again direct itself into the pipe, restoring

Mr. Kruse.

the normal pressure, first in section n, next to the reservoir, and then, in rapid succession, in the other sections $(n - 1), \ldots .4, 3$, etc., until, when the normal pressure reaches the gate, we have once more the conditions which existed just before the gate was closed, viz., the normal pressure is restored and the water is moving toward the gate with the original velocity, v.

"We have now followed these pulsations of pressure (with the accompanying transformations of energy and flow of water into and back from the pipe) through a complete cycle of four movements, each extending through the length of the pipe. For convenience, we may consider two successive movements of this kind as a 'round trip' through the pipe.

"The gate remaining closed, the whole process is now repeated in a second cycle, which, in turn, is followed by a third, and so on, the amplitude of the pressure vibrations gradually diminishing (because of friction) until the pipe and the water come to a state of rest.

"But although the intensity of the pressure becomes gradually less, the time required for each cycle remains constant for all repetitions.

"This propagation of pressure, consisting of its transmission through all points of the length of the pipe, each point successively repeating the same periodical movements, is, in its nature, simply a case of wave motion, like that of a sound wave.

"The velocity of wave propagation is independent of the intensity of the pressure, and depends only upon the properties of the medium through which the propagation takes place—in the case of water-hammer, upon the elasticity of the water and of the pipe."

Mr. Gibson has applied this theory to slow-closing gates by considering that each infinitesimal movement of the gate is instantaneous and produces a small rise of pressure which travels through the penstock in the same manner that a wave produced by total instantaneous closing would travel. These small gate movements occur in succession and produce in turn their pressure waves which travel back and forth through the penstock. An algebraic sum of these waves gives the rise of pressure existing at any time during the closing stroke.

The writer is of the opinion that the pressure existing throughout the length of the pipe for a slow-closing gate varies almost directly from a maximum at the gate to zero at the point of relief, except where the time of closing is less than $\frac{2\,L}{a}$.

In Fig. 11, the pressure existing at four points on the penstock has been plotted. The magnitude of the individual pressure waves is not to scale, but their values are shown for the particular example taken. The pressure existing at a point $\frac{L}{2}$ from the gate is approximately one-half the pressure existing at the gate. The pressure existing at a point $\frac{3\,L}{4}$ from the gate is approximately one-quarter of the pressure existing at the gate.

Mr. Kruse.

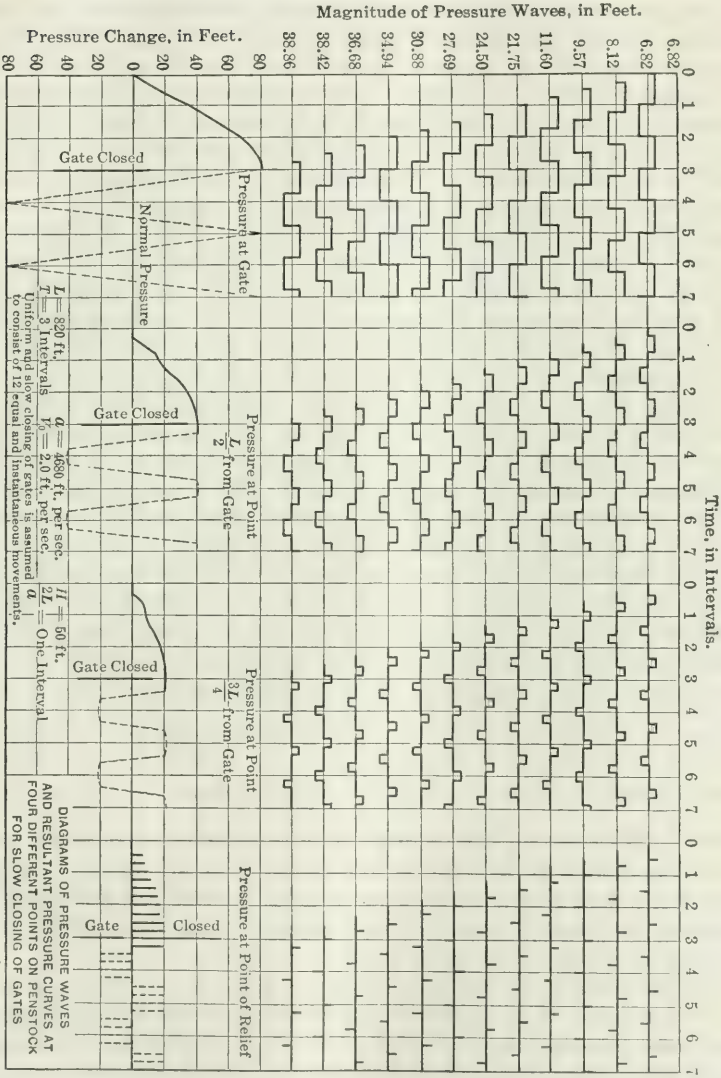
Magnitude of Pressure Waves, in Feet.
Time, in Intervals.
Pressure Change, in Feet.
Gate Closed
Normal Pressure
Pressure at Gate
Pressure at Point L/2 from Gate
Pressure at Point 3L/4 from Gate
Pressure at Point of Relief
Gate
Closed
a = 4680 ft. per sec.
V0 = 2.0 ft. per sec.
H = 50 ft.
L = 820 ft.
T = 3 Intervals
2L/a = One Interval
Uniform and slow closing of gates is assumed to consist of 12 equal and instantaneous movements.
DIAGRAMS OF PRESSURE WAVES AND RESULTANT PRESSURE CURVES AT FOUR DIFFERENT POINTS ON PENSTOCK FOR SLOW CLOSING OF GATES

FIG. 11.

Mr. Kruse.

The diagram showing the pressure existing at the point of relief consists of a series of instantaneous rises above normal. On account of the assumption of twelve instantaneous movements of the gate, the pressure at the point of relief must of necessity show twelve instantaneous rises of pressure, as each wave travels undiminished to the point of relief. If twenty-four instantaneous movements were chosen, there would appear twenty-four instantaneous rises of pressure in the same time, but of only about half the magnitude. Hence, in the limit of an infinite number of instantaneous movements of the gate, or uniform slow-closing, the pressure at the point of relief becomes zero. This must be true, because the duration of time of any rise of pressure at the point of relief is zero. Wherever rises of pressure have actually been recorded at the point of relief on a penstock, they are probably due to an instantaneous movement of the gate during the stroke, or to some other cause, such as a sudden elastic deformation of the waterway, which would have the same effect.

In the translation of Professor Joukovsky's work there are shown tables of pressure rises at different points on the pipes for different velocities destroyed. In every case the pressure waves travel practically undiminished almost to the point of relief. The readings taken at the station nearest the point of relief, however, show a reduction in pressure of from 60 to 80 per cent. This is very significant to the writer, as an indication that the pressure traveled undiminished to a certain point and then gradually was reduced to zero at the point of relief. Although Professor Joukovsky experimented with very rapidly closing gates, as nearly instantaneous as possible, obviously a certain fraction of a second must elapse while the gate is closing. When the time of closing is less than one interval, $\frac{2L}{a}$, the resultant pressure will travel undiminished to a point which is $\frac{Ta}{2}$, in feet, from the point of relief (where T is equal to the closing time, in seconds). From this point it will gradually be reduced to zero at the point of relief. Granting that some short period of time must elapse during the closing of the gates in Professor Joukovsky's experiments, his actual pressure readings seem to verify the writer's claim.

In the example taken, the duration of closure is assumed to be three intervals. The diagrams and curves have been plotted beyond the three intervals to show the fluctuations of pressure which take place after the gates are closed. Experiments should show that the subsequent fluctuations after the gates are closed are quickly reduced in intensity by internal friction, but this friction should have no effect on the speed of propagation of the pressure waves. Internal friction has of necessity been omitted in these computations, as it is an unknown quantity.

Any other example than that shown by Fig. 11 may be chosen, and the pressure waves may be computed by Mr. Gibson's formulas. A series of diagrams may then be plotted, similar to those shown by the writer, and the pressures may be computed for any point on the penstock. Mr. Kruse.

In making the computations for the magnitude of the pressure waves, it is necessary to determine first the number of intervals of time $\left(\text{equal to } \frac{2L}{a}\right)$ contained in the total closing time. The gate movement may then be divided into a number of instantaneous movements equal to the number of intervals. Computations carried out on this basis will give correct points on the pressure-rise curve at the end of each interval. In the event of the closing time being only a few intervals, it is desirable to subdivide the gate movement further in order to develop the curve during the intervals. It is an interesting fact, however, that such further subdivision of gate movement serves only to develop the shape of the curve during the intervals, and has no effect on the values at the end of each interval. In making studies of this character, it is often easier instead of using Mr. Gibson's formulas, to utilize the system of arithmetic integration, which will give correct results, although involving trial-and-error methods.

In order to compute the pressure-rise curve at points on the penstock other than at the gate, the gate movement must be divided into a number of divisions sufficient to cause the pressure waves to overlap. Thus, at least two divisions per interval must be used to compute the curve at a point half way up the penstock, at least four divisions per interval for a point three-quarters of the way up the penstock, and at least eight divisions per interval for a point seven-eighths of the way up the penstock. Referring to Fig. 11, where four divisions per interval were chosen, it is obvious that the pressure beyond the point, $\frac{3L}{4}$ from the gate, cannot be computed without a greater subdivision of the gate movement.

In this discussion the writer has endeavored, principally, to point out the trend of argument and application of existing data used in proving the theoretic correctness of Mr. Gibson's method of handling water-hammer problems. The method seems to be fundamentally sound, and deserves the careful thought of engineers.

Mr. Halmos.

EUGENE E. HALMOS,* ESQ. (by letter).—Ever since the publication of the paper† entitled "Penstock and Surge-Tank Problems" by Minton M. Warren, Assoc. M. Am. Soc. C. E., the writer has hoped that some one, preferably a member of the Society, would point out

* Designing Engineer, Barclay Parsons and Klapp, New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 238.

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the injustice done to one of the greatest hydraulic engineers of the present age by the form of, and the comments appended to, Mr. Warren's presentation of what he termed "Alliévi's formula."

Unfortunately, Mr. Warren has not been corrected. This formula was accepted by American engineers as representing Alliévi's solution of the problem of water-hammer in pipe lines, and, consequently, it has been severely criticized and misinterpreted.

The value of Mr. Gibson's excellent paper is also seriously impaired by the references, both in the text and in the diagrams, to "Alliévi's formula" as taken from Mr. Warren's paper, and the writer would suggest that, after perusal of the following, Mr. Gibson omit all reference to Alliévi, except to state that his solution agrees, in every particular, with that of Alliévi.

The writer proposes to submit a brief summary of Mr. Alliévi's formulas, which, in his judgment, represent a complete solution of water-hammer problems as regards determination of the pressure at the gate or at any other section of the conduit, at any instant, and under any conditions of closure and physical characteristics of the plant. It is more general than Mr. Gibson's solution, inasmuch as it also includes the determination of the pressure variations after the stopping of the gate movement, and as it gives a method of estimating the interval in which the pressure maximum will occur.

As the present paper and the previous ones deal almost exclusively with the case of water-hammer during the closure of gates, and with the variation of the pressure at or near the gates, the writer will only quote Alliévi with regard to formulas applicable to such cases. The writer, however, intends to translate Alliévi's book into English, which, it is hoped, will be of great help to designers of hydraulic plants and apparatus.

In the following, the formulas representing the relation of time and pressure are given for three phases, namely:

the first phase, for which $t \leqq \frac{2\,L}{a}$;

the second phase, for values of t beginning with $t = \frac{2\,L}{a}$ and ending at T;

and the third phase, $t > T$.

Notation.—

a = velocity of propagation of pressure along pipe;

c_0 = uniform velocity of water in pipe before beginning of gate closure;

C = variable velocity of water in pipe at any instant during gate closure;

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u_0 = uniform velocity of water through gate orifice before beginning of closure;
u = variable velocity of water through gate orifice during gate closure;
t = time, in seconds, recorded from beginning of gate movement;
T = duration of gate movement, assumed to be continuous;
L = length of pipe conduit;
y_0 = pressure head on pipe line at gate before closing begins;
y = variable head at gate due to closure.

A.—General Formulas.

(1) *Variation of Pressure Head During and After the Closing Motion of the Gates.—*

First phase, $t \leq \frac{2\,L}{a}$.

For this interval, the pressure head, y, at any instant can be computed from the equation

$$y^2 - 2\,y\left(H + \frac{a^2\,\psi^2\,(t)}{g}\right) + H^2 = 0 \quad \ldots\ldots\ldots\ldots (1)$$

in which $$H = y_0 + \frac{a\,c_0}{g}$$

and $\psi\,(t)$ is the ratio of the instantaneous area of discharge to the (assumed) constant area of the penstock, that is, $\psi\,(t) = \left(1 - \frac{t}{T}\right)\frac{c_0}{\sqrt{2\,g\,y_0}}$ the value of $\psi\,(t)$ being taken at the instant for which the pressure head, y, is calculated.

It will be noted that if the gate is closed in a time, $T \leq \frac{2\,L}{a}$, $\psi\,(T) = 0$, and the equation reduces to

$$y^2 - 2\,H\,y + H^2 = 0$$

$$y = H = y_0 + \frac{a\,c_0}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

which shows that in such a case the maximum pressure head is independent of the actual length of the time of closure.

Of the two roots of Equation (1) one is greater, the other is smaller, than H. This latter is the true value of the pressure head at any instant.

Second phase, $\frac{2\,L}{a} \leq t \leq T$.

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During this period of the gate movement, the following equation applies for the determination of the pressure head at any instant:

$$y^2 - 2\,y\left(H - 2\,f + \frac{a^2\,\psi^2\,(t)}{g}\right) + (H - 2\,f)^2 = 0 \ldots\ldots (3)$$

Of the two roots of Equation (3), that giving the smaller value should be used.

It will be noted that this equation differs from Equation (1) only in so far that, instead of H in the latter, $(H - 2\,f)$ appears in Equation (3).

The values of the function, f, should be determined in the following manner:

For the interval, $0 < t < \frac{2\,L}{a}$, $y - y_0 = F\,(t)$, y being calculated from Equation (1).

The velocity, C, in the conduit (at the gate) can be computed from the equation

$$y + \frac{a}{g}\,C = H$$

for this interval.

For the interval, $\frac{2\,L}{a} < t \leqq \frac{4\,L}{a}$, $f = F\left(t - \frac{2\,L}{a}\right)$; in other words, the values of $F\,(t)$ found in the first interval should be substituted for f in Equation (3) to obtain the value of y for every subdivision of the second interval. Then, the corresponding values of the velocity, C, may be found by $y + \frac{a}{g}\,C = H - 2\,f$, and the value of $F\,(t)$ for the second interval by $F\,(t) = \frac{a}{g}\,(c_0 - C) - f$.

For the third interval, $\frac{4\,L}{a} < t \leqq \frac{6\,L}{a}$, the values of $F\,(t)$ so found for the subdivisions of the second interval should be substituted for f in Equation (3) to get y, and so on, for all subsequent intervals until the gate has stopped moving or has become entirely closed.

Third phase, $t > T$.

If the movement of the gate is stopped at an instant such that $t_1 < \frac{2\,L}{a}$, the pressure head, y, remains constant until $t = \frac{2\,L}{a}$. From this instant on, or from the time of the stoppage, in the case, $t_1 > \frac{2\,L}{a}$, there is developed a hydrodynamic phenomenon in the form of an asymptotical approach of the pressure head, y, to the new constant head, y_0, of uniform flow. There are three different cases.

First.—If $a\ \psi\ (t_1) > \frac{u_0 + u_1}{2}$, in which case y approaches y_0 without oscillations.

Second.—If $a\ \psi\ (t_1) < \frac{u_0 + u_1}{2}$, in which case y approaches y_0 through oscillations of diminishing amplitude.

Third.—If the gate is entirely closed, $\psi\ (t_1) = 0$, the pressure head (disregarding the dampening effect of hydrodynamic friction) oscillates indefinitely, with a constant amplitude, between the value of y, which occurs at the instant of closing, and $2\ y_0 - y_1$, the amplitude being $y_1 - y_0$.

(2) *Estimating the Phase in Which the Maximum Pressure Will Occur.*—Assuming a linear closure of the gates:

(a) If $a\ c_0 < 2\ g\ y_0$, then the pressure head at the end of the first phase $\left(t = \frac{2\ L}{a}\right)$ will always be greater than the average maximum pressure (found by Equation (4)) occurring during the second phase.

(b) If $2\ g\ y_0 < a\ c_0 < 3\ g\ y_0$, the maximum pressure head occurs either at the end of the first phase, or during the second phase, according to whether the closing time, T, is smaller or greater than

$$\frac{a\ c_0 - g\ y_0}{a\ c_0 - 2\ g\ y_0} \cdot \frac{L}{a}.$$

(c) If $a\ c_0 > 3\ g\ y$, then the maximum pressure will always occur during the second phase and will be greater than that at the end of the first phase.

Equations (1), (2), and (3) are the general formulas derived by Alliévi, by the help of which complete pressure-time curves can be worked out for any assumed conditions of T, L, a, c_0, and y_0. The writer is very much pleased to state, that, by the use of these equations, he was able to check accurately all the curves presented by Mr. Gibson. He wishes to compliment the author in having found, quite independently, and by a simpler mathematical method than Alliévi's functional derivation, the correct formulas for the representation of pressure-time curves and related phenomena.

B.—Special Formulas.

It has been found by experiment, and by working out pressure-time curves for a great number of cases actually met in practice, that, on the assumption of linear gate closure:

(d) The pressure behind the gate remains practically constant for $\frac{2\ L}{a} < t \leq T$ (y is not a function of time or $\frac{d\ y}{d\ t} = 0$ between

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these limits) and, therefore, for this period, the pressure is independent of the elastic qualities of the water and the conduit, in other words, independent of (a);

(e) The pressure is distributed in a linear way along the pipe ;

(f) The velocity of the water, C, is the same at any instant at any section of the conduit $\left(\frac{d\,c}{d\,x} = 0\right)$.

Taking into consideration the observations noted under (d), (e), and (f), a good average value of y maximum during the second phase can be obtained by solving the equation

$$Z^2 - Z\,(n^2 + 2) + 1 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where $$Z = \frac{y_{max.}}{y_0} \text{ and } n = \frac{c_0\,L}{g\,T\,y_0}.$$

This equation is identical with Equation (C) in Mr. Warren's paper. In order that it should be valid, it is necessary:

(1) That T be greater than $\frac{2\,L}{a}$;

(2) That the gate movement (the rate of reduction of the area of the outlet) be a linear function of the time;

(3) That, in case of partial closure, that value of T should be used which would be obtained if the gate should be completely closed at the same rate of speed;

(4) That the physical data, as L, a, y_0, and c_0, shall conform to those ordinarily met in practice.

In order that Equation (4) should produce the maximum pressure head obtaining during the whole movement of the gate, it is necessary that $a\,c_0$ shall take the values defined under (b) and (c).

It is evident that Mr. Warren committed a serious error by omitting the publication of Alliévi's general Equations (1), (2), and (3), and by omitting the proper definition of Equation (4) and the limitations of its applicability as defined by Alliévi.

The only ambiguity which may occur in using this formula is the lack of a better definition of Condition (4). This is extremely difficult, on account of the great number of variables entering into the problem. The writer, however, proposes, without being able to give strict analytical proof at the present time, that Equation (4) should be used only, if

(α) $$\frac{L\,c_0}{g\,T\,y_0} < \frac{1}{2}\sqrt{\frac{a\,c_0}{y_0\,g} + \frac{y_0}{y_0 + \frac{a\,c_0}{g}}} - 1;$$ and,

(β) $$a\,c_0 > 2\,g\,y_0.$$

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As L, c_0, a, and y_0 are dependent on rather unalterable physical conditions, if Condition (β) is satisfied, Condition (α) can be fulfilled by determining T according to the proposed formula.

If, besides Conditions (α) and (β), Conditions (b), (c), (1), and (2) are also fulfilled, Equation (4) will give the absolute maximum head at the gate during the movement of the apparatus.

It will be found that,

$$\text{for } c_0 \geqq 10 \text{ ft. per sec.}$$

$$y_0 = \text{between 100 ft. and 500 ft.}$$

$$T \geqq 6 \text{ intervals,}$$

all the foregoing conditions are fulfilled, which means that, in most practical cases, Equation (4) actually furnishes the maximum pressure head at the gate.

If the elasticity of the water and the conduit is neglected for the whole time of the movement of the gate, then, for the conditions for which Equation (4) is applicable, the pressure-time curve can be platted from the equation:

$$\log. \frac{\left(\sqrt{u_0^2 + u_R^2} + u - u_R\right)\left(\sqrt{u_0^2 + u_R^2} - u_0 + u_R\right)}{\left(\sqrt{u_0^2 + u_R^2} - u + u_R\right)\left(\sqrt{u_0^2 + u_R^2} + u_0 - u_R\right)} = \frac{\sqrt{u_0^2 + u_R^2}}{u_R} \log. \frac{T}{T - t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which $u_R = K \frac{L}{T}$, where K is the ratio of the area of the gate opening at $t = 0$ to the area of the pipe, and

$$u_0 = \sqrt{2\,g\,y_0}, \quad u = \sqrt{2\,g\,y}.$$

This curve will have a maximum at $t = T$ if Conditions (b) and (c) are fulfilled, and will have a practically horizontal position in the second phase if Condition (α) is observed. It will differ very little from the true curve of pressure variations, and is applicable to practically the whole range of actual water-power problems.

It will be noted that Equations (4) and (5) are identical with Equations (6) and (7) given by Mr. R. D. Johnson in his discussion* of Mr. Warren's paper. Great credit is due to Mr. Johnson for having grasped correctly the essential features of this difficult subject, and for his wonderful mathematical skill in solving by independent methods the intricate problem of water-hammer.

In conclusion, the writer wishes to observe, that it would be desirable to encourage the use of Allióvi's, Gibson's, and Johnson's formulas for

* *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 280.

Mr. Halmos. water-hammer, and to reject and discourage the use of other formulas, some of which, such as Mr. Warren's, are never true, not even accidentally.

Mr. Johnson. R. D. JOHNSON,* ESQ. (by letter).—Mr. Gibson's remarkable paper deserves the closest attention on the part of all who are interested in the water-hammer problem.

He remarks that the theory has not been experimentally confirmed, but it is so obviously complete and correct as scarcely to need such confirmation to establish strong faith in its accuracy.

The writer has been especially interested in a completely correct treatment of this subject, in order to be able to fix the limits of K within which his pressure-time curve, previously presented to the Society, is sufficiently accurate for practical purposes.

It is found, fortunately, that this smooth curve,† $\log. x = n \log. \frac{T}{T - t}$, is applicable with much precision to such a wide range of conditions as to justify its use for practically all the water-hammer problems arising in connection with the design and operation, under ordinary heads, of water turbines, when properly regulated.

By differentiating Mr. Gibson's equations, it is possible to express rigidly correct values of the tangents to the true pressure-time curve at (a), the beginning of gate closure, (b), the end of the first interval, and (c), the beginning of the second interval.

Using Mr. Gibson's nomenclature, with omission of subscripts, that is, putting $R_0 = R$, and $S_{T_1} = S$, and $H_0 = H$, these tangents may be expressed as follows:

$$\text{tan. } (a) = \frac{2\,R\,H}{(R + 2\,H)\,T}$$

$$\text{tan. } (b) = \frac{R^2\,(T - T_1)}{H\,T^2}\left\{\frac{S + 2\,R + 2\,H}{\sqrt{S\,(S + 4\,R + 4\,H)}} - 1\right\}$$

$$\text{tan. } (c) = \frac{R^2\,(T - T_1)}{H\,T^2}\left\{\frac{S\,(Z+1) + 2\,R + 2\,H}{\sqrt{S\,(S + 4\,R + 4\,H)}} - (Z + 1)\right\}$$

where $Z = \frac{4\,H^2\,T}{R\,(R + 2\,H)\,(T - T_1)}$.

It is clear that the effects of vibration in the water column, due to elasticity, tend to disappear when the conditions are such that tan. (b) and tan. (c) approach equality; also, if the change in the rate of pressure rise at the beginning of the second interval, due to the effect of

* New York City.

† The derivation of this curve, Equation (7), is given in *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 280.

the returning wave, is less than the rate at which the pressure begins to rise when the gate starts to move, it is evident that the conditions are such that the effect of elasticity is rapidly dying out, and that it should be scarcely appreciable during the remainder of the closing period. Mr. Johnson.

If, then, tan. (b) — tan. (c) < tan. (a), the writer's Equation (7), above referred to, or the maximum value of h, given by the so-called Alliévi formula, may be used with much precision.

This may be shown to be the case when,

$$K = \frac{2}{\sqrt{3\,r^2 + 1} - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where r is the ratio, $\frac{T - T_1}{T}$; and $K = \frac{h}{H}$.

Formula (1) marks the lower limit for values of K, below which it is not safe to use Equation (7). To find the upper limit of safety, for values of K, it is noted that, as H decreases, the pressure-time diagram passes through a transition stage where an ogee form of curve most closely approaches to a straight line, and, that thereafter, as H continues to decrease, Equation (7) begins to produce a curve sharply upturning and curving one way, for the most part, as its extremity reaches toward infinity, thus leading to serious errors in its results.

By trial, assisted by arithmetic integration, it has been noted that, for large values of n, the head may be reduced to about one-half the value which produces the approximate straight-line diagram.

To find this point we may equate the value of $h_{max.}$ in the so-called Alliévi formula to the value of the pressure rise found at the end of a straight-line pressure-time diagram, enclosing the same impulse area, and solve for the value of H.

In this way it is found that

$$K = 3n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

(where n is the closing time expressed in intervals), and Equation (7) may be used so long as

$$K = \text{or} < \frac{6\,n^2}{n + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

which, for large values of n, is nearly twice as great as the value given by Formula (2), but which becomes equal to 3, as it necessarily must, when n becomes unity.

Formula (3) marks the upper limit for values of K, above which it is not safe to use Equation (7). Therefore, the smooth logarithmic curve given by Equation (7) may be applied only when K lies between the values given in Formulas (1) and (3), and preferably, also, when n is not less than three intervals.

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Fig. 12 shows that Equation (7) covers a very wide range of conditions, and, furthermore, in the design of a water-turbine installation, the values of V and n should, when possible, be adjusted so that Equation (7) will be correctly applicable; and particular attention should be given to the limitations set forth in Formulas (1) and (3), in order that the speed of the water unit may be regulated most effectively.

The expressions for the tangents, (a), (b), and (c), are rigidly accurate, and form a substantial basis for further possible study; for example, if tan. (c) be equated to zero, the limiting value of n may be found when the maximum pressure rise occurs at the end of the first interval. This is thus found to be the case, always, when

$$n > \frac{4\ K^2 + 8\ K}{12 - K^2 - K^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

It is seen that this expression is positive and finite only so long as $K < 2$; therefore, K must always be less than 2, in order that the maximum rise shall occur at this point. It does not follow that this will be the case merely because $K < 2$, unless Formula (4) also applies.

In support of Mr. Gibson's work by experimental observation, it may be pointed out that much information is available for cases which occur within the limits here set forth, and, inasmuch as his complete work incidentally includes such cases, it may safely be predicted that no error in theory will be discovered by experiment.

The writer, himself, has confirmed Equation (7) a great many times, under a variety of conditions, in the past twenty years, and, because of such confirmation, he was not led, from practical necessity, to a more thorough investigation of the theory.

In conclusion, the writer wishes to urge those interested to a further study of the limits here graphically presented, with the idea either of confirming or improving them, because it is not profitable to resort to the very tedious processes incident to the methods presented by Mr. Gibson, except when a comparatively simple formula is inapplicable. The chart, Fig. 12, indicates that this is very rarely the case, and practically never need be, with proper care in the design and operation of any water system.

It is interesting to note that Mr. Gibson's methods may be used to work out a system of perfectly general cases, thus enlarging, indefinitely, the scope of such diagrams as he has presented, which are based, merely, on a specially selected set of data.

This is made possible by simply substituting in his scale of ordinates, values of the ratio $(h_{max.}) \div H$ in place of $(h_{max.})$, and substituting for the values of H, written on the curves, the values of K.

This trifling change, it will readily be seen, will make such diagrams applicable to an infinite variety of special cases.

In this manner, the chart, Fig. 13, has been roughly prepared, and, for the range of values covered, it is not necessary to resort to formulas. Mr. Johnson

All combinations of a, V, and H which produce a constant value of K must invariably result in a definite single value of the ratio

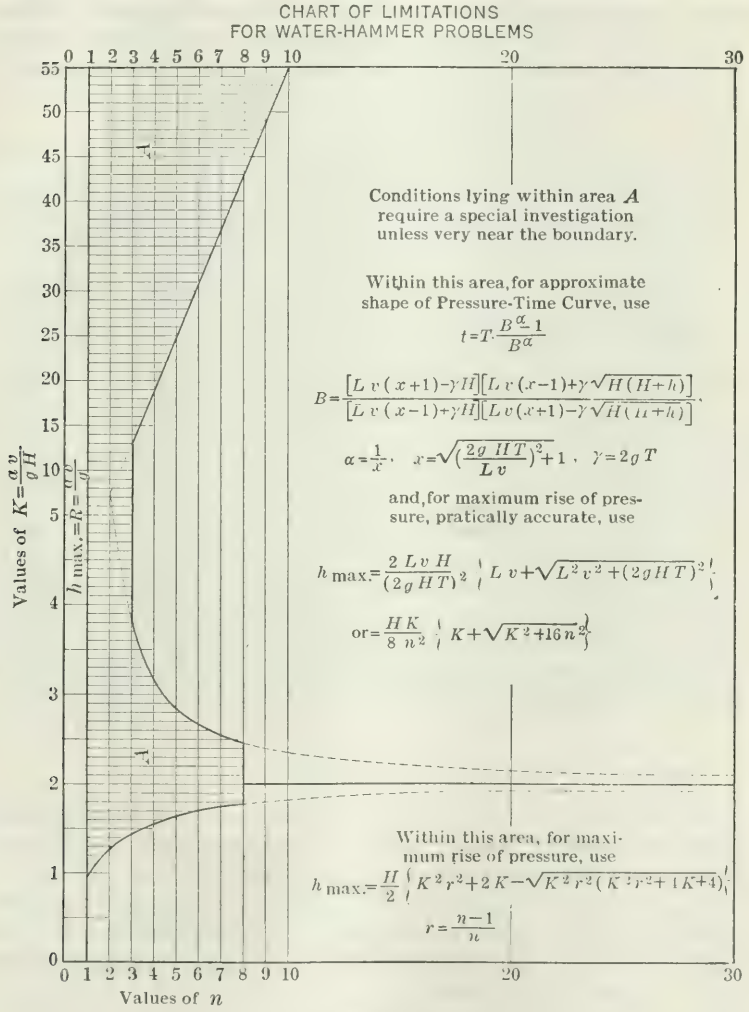
CHART OF LIMITATIONS
FOR WATER-HAMMER PROBLEMS
Values of $K=\frac{a\,v}{g\,H}$
$h\,\text{max.}=R=\frac{(\Delta p)_{max.}}{w}$
Values of n
Conditions lying within area A require a special investigation unless very near the boundary.
Within this area, for approximate shape of Pressure-Time Curve, use
$t=T.\frac{B^{\alpha}-1}{B^{\alpha}}$
$B=\frac{[L\,v\,(x+1)-\gamma H][L\,v\,(x-1)+\gamma\sqrt{H(H+h)}]}{[L\,v\,(x-1)+\gamma H][L\,v\,(x+1)-\gamma\sqrt{H(H+h)}]}$
$\alpha=\frac{1}{x}$, $x=\sqrt{(\frac{2g\,HT}{L\,v})^2+1}$, $\gamma=2g\,T$
and, for maximum rise of pressure, pratically accurate, use
$h\,\text{max.}=\frac{2\,L\,v\,H}{(2g\,HT)^2}\left\{L\,v+\sqrt{L^2v^2+(2g\,HT)^2}\right\}$
or $=\frac{H\,K}{8\,n^2}\left\{K+\sqrt{K^2+16\,n^2}\right\}$
Within this area, for maximum rise of pressure, use
$h\,\text{max.}=\frac{H}{2}\left\{K^2r^2+2K-\sqrt{K^2r^2(K^2r^2+4K+4)}\right\}$
$r=\frac{n-1}{n}$

FIG. 12.

$(h_{max.}) \div H$ for each selected value of n. Therefore, in fact, each point of the chart covers, theoretically, an infinite variety of conditions.

Mr. Johnson.

In the use of the chart or simple formulas to replace the accurate but tedious methods, it should be borne in mind that it is not ordinarily practically necessary that the agreement should be perfect, because, as Mr. Gibson has shown, the shape of the curve is so sensitive to the vagaries of the gate motion, in closing, as to make it almost out of the question to reproduce, experimentally, any definitely selected shape of curve; so that, after all, the element of judgment must enter, as usual.

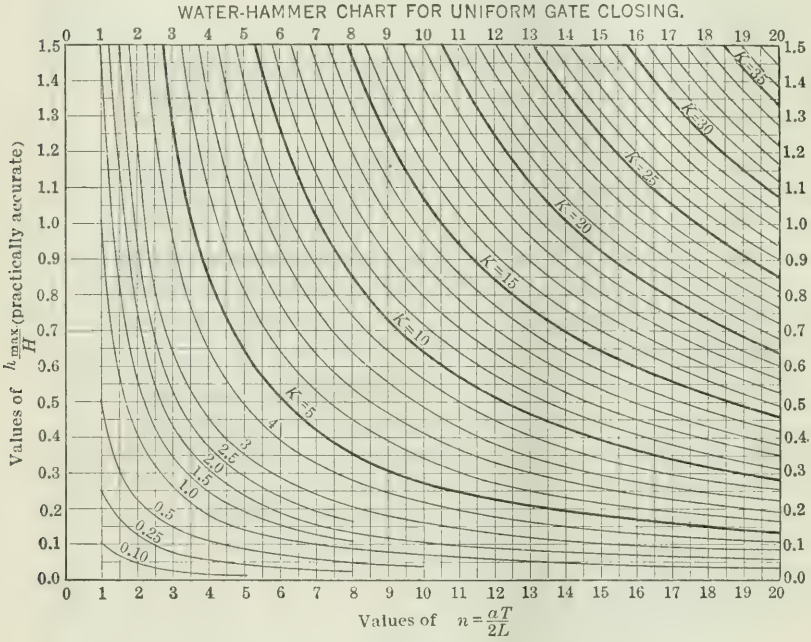

FIG. 13.

The effects of velocity head and friction head have been neglected in the preparation of Figs. 12 and 13. These factors are nearly always of minor importance, as affecting the rise of pressure, but, nevertheless, they may be taken into account by means of arithmetic integration in special cases where great accuracy is thought to be warranted. These effects, at any rate, cannot be included in a usable formula, covering the whole range of the charts.

For the benefit of those who are interested in the phenomena of water-hammer only sufficiently to desire a quick, comprehensive grasp of the derivation of the various formulas from fundamental principles, it seems worth while to present a simple analysis based on a straightforward hypothesis which has been generally accepted as applying

Mr. Johnson.

to elastic columns, but which has not, so far as the writer knows, been directly and clearly stated. This is attempted in the following language:

When a varying pressure is applied against the end of an elastic column, the change of velocity of that end, in the time required for the pressure wave to traverse the length of the column and return, is proportional to the arithmetic mean of the two pressures which there exist at the two extremities of this "interval" of time; and the impulse produced by the product of this average pressure and the time, $\frac{2L}{a}$, is equal to a change in momentum corresponding to the total mass of the column multiplied by the change in velocity of its end, adjacent to the point of application of the pressure.

This is a perfectly general statement applicable to any "interval," $\frac{2L}{a}$, of time during the acceleration (negative) of the elastic column.

If time is measured in "intervals" instead of seconds, we may write $\frac{2L}{a} \times n$ (intervals) $= t$ and $\frac{2L}{a} \times N = T$, where N is the total number of intervals (either a whole or mixed number) required to bring the end of the elastic column to a state of rest, and n, also either a whole or a mixed number, corresponding to any number of seconds, t.

With particular reference to a column of water arrested by the closure of a gate, we may designate the instantaneous pressures (h) above normal (H_0), at the extremities of any interval of time during closure of the gate, as h_{n-1} and h_n, respectively, and may write the general expression corresponding to the hypothesis (omitting the area of the pipe, assumed constant, and cancelling the common factor denoting the specific weight of the fluid), as follows:

$$\frac{h_{n-1} + h_n}{2} \times \frac{2L}{a} = \frac{L}{g}(V_{n-1} - V_n)$$

or, more simply,

$$h_{n-1} + h_n = \frac{a}{g}(V_{n-1} - V_n) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This is the simple relation of which Mr. Gibson took advantage in working out his arithmetic tables, and to which he refers at the end of the paragraph on page 719.

During the first interval, V_{n-1} becomes the initial velocity, V_0, and h_{n-1} becomes $h_0 = 0$, so, for values of n equal to or less than unity (or for values of $t \leq \frac{2L}{a}$), we may write:

$$h_{n-1} = \frac{a}{g}(V_0 - V_n) \ldots\ldots\ldots\ldots\ldots\ldots (1A)$$

Mr. Johnson.

If the gate is completely closed in one interval or less, we have the maximum possible rise of pressure; for, in that case, V_n becomes zero and we may write:

$$h_{max.} = \frac{a}{g} V_0$$

which is the famous formula of Joukovsky.

Thus, it is clear how all these fundamental equations are derivable from the one general hypothesis which, itself, reverts to the second law of Newton.

Equation (1) comprises the essential features of the water-hammer problem for fast or slow-closing gates, and a particular case may be worked out for any known or assumed character of closure, with respect to time, by expressing the simple relation between the quantity of water discharging at any time through the gate-opening existing at that time, and the total pressure head which drives the water through it. For example, if ϕ_n represents the proportion, at any time, of full gate-opening, the area of which is b, and the coefficient of discharge is c, we have (neglecting heads due to velocity and friction),

$$\phi_n b c \sqrt{2 g (H_0 + h_n)} = A V_n \quad \ldots\ldots\ldots\ldots\ldots (2)$$

and

$$\phi_0 b c \sqrt{2 g (H_0 + h_0)} = A V_0 = b c \sqrt{2 g H_0} \quad \ldots\ldots\ldots (3)$$

where A is the area (constant) of the pipe.

Dividing Equation (2) by Equation (3),

$$\phi_n V_0 \sqrt{1 + \frac{h_n}{H_0}} = V_n$$

or, putting $\frac{h_n}{H_0} = P_n$, we have,

$$V_n = \phi_n V_0 \sqrt{1 + P_n} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

and

$$V_{n-1} = \phi_{n-1} V_0 \sqrt{1 + P_{n-1}} \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Subtracting Equation (4) from Equation (5),

$$V_{n-1} - V_n = V_0 \left(\phi_{n-1} \sqrt{1 + P_{n-1}} - \phi_n \sqrt{1 + P_n}\right) \ldots (6)$$

Thus, we have another expression proportional to ΔV, which may easily be made simultaneous with Equation (1), which latter may be rewritten, as follows:

$$V_{n-1} - V_n = \frac{g H_0}{a} (P_{n-1} + P_n) \quad \ldots\ldots\ldots\ldots\ldots (7)$$

Equating the second terms of Equations (6) and (7), we have the general water-hammer formula for closing gates, as follows:

$$P_{n-1} + P_n = K\left(\phi_{n-1} \sqrt{1 + P_{n-1}} - \phi_n \sqrt{1 + P_n}\right) \ldots\ldots (8)$$

where $K = \frac{a V_0}{g H_0}$.

The successive values of ϕ with respect to time are quite independent of the other factors of the equation, and they may be all

predetermined from an arbitrary or assumed character of gate closure, with respect to time. This may be expressed in the form of an independent curve plotted, for example, from some known motion of a particular gate with respect to time. Mr. Johnson.

If the gate closure is uniform, as Mr. Gibson has assumed in deriving his formulas, then the equation for ϕ may be written out, for subsequent use, or plotted as a straight line on a piece of cross-section paper. In this special case, the equation would be,

$$\phi_n = 1 - \frac{n}{N}, \text{ or } = 1 - \frac{t}{T}$$

as he states.

During the first interval, Equation (8) may be simplified to read,

$$P_{n \leqq 1} = K\,(1 - \phi_n \sqrt{1 + P_n}) \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

In order to "get a start" in computing the successive values of P, one interval apart, the first value of $\phi_{n \leqq 1}$ may be selected from the gate-opening curve at any convenient proportion of the first interval, and $P_{n \leqq 1}$ found from Equation (9).

Then, and thereafter, the successive values of ϕ taken from the curve, one interval apart, may be substituted in Equation (8) and the successive values of P_n found from the known values of P_{n-1} as the computations progress. If the complete pressure-time curve is desired, several different proportions of the first interval must be selected and the corresponding values of ϕ introduced first in Equation (9).

Equations (8) and (9) may be rearranged, of course, in the form of the solution of quadratics, separating out the values of P_n on one side, as Mr. Gibson has done, or these values may be solved by trial as he has done in making up his tables. The result is, naturally, the same in either case, and one method is no more nor less accurate than the other.

It is interesting to note the physical story told by a slight rearrangement of Equation (1) when $2h_{n-1}$ is subtracted from both sides. It then appears in this form:

$$h_n - h_{n-1} = \frac{a}{g}\,(V_{n-1} - V_n) - 2\,h_{n-1} \ldots\ldots\ldots (10)$$

Now, the term $\frac{a}{g}\,(V_{n-1} - V_n)$ represents the "natural rise" in pressure in a pipe of indefinite length, or at least so long that returning waves do not interfere with the natural rise during the time taken for the velocity to change the stated amount. This is the case in the first interval as shown by Equation (1*A*).

We know from the principles laid down by Joukovsky that a supernormal pressure wave initiated at the beginning of any "interval" will become subnormal upon its return at the end of the "interval" of time,

Mr. Johnson.

or will be, so to speak, turned "upside down", and hence that its reducing effect on the pressure which would otherwise exist at the end of such "interval" is twice as great as the original supernormal value. This accounts for the term $2h_{n-1}$, and we may write another general hypothesis which is adequate for the statement of Equation (1), as follows:

The rise of pressure during any interval, $\frac{2L}{a}$, of time is equal to the "natural" or undisturbed rise, reduced by twice the value of the total pressure (above normal) which existed at the beginning of such "interval".

There are probably many ways in which to express the philosophy of Equation (1), and each may select the one best suited to his mind. The following is one of them:

If the pressure, h_{n-1}, should remain constant throughout the "interval" which it precedes, it is known, from the nature of an elastic column, that each and every particle of water would suffer the same diminution in velocity during the ensuing whole "interval" and, hence, that the change in velocity of the column, as a whole, would be faithfully recorded by the equal change at the end of the column, next to the gate. Call this portion of the velocity change, $\Delta_1 V$. Then, from the relation of impulse to change of momentum, we may write,

$$h_{n-1}\left(\frac{2L}{a}\right) = \left(\frac{L}{g}\right)\Delta_1 V, \text{ or, } 2h_{n-1} = \left(\frac{a}{g}\right)\Delta_1 V$$

Now, any pressure added later in the interval under consideration, could have time, during such interval, to distort only a portion of the water column (of total round-trip length, $2L$), and the change in velocity produced at the end of the column by any such increment of pressure, Δh, may be derived as follows:

The mass affected in the time, dt, is proportional to adt.

This mass, multiplied by its acceleration, equals the increment of force to produce the acceleration. Therefore,

$$dh = \left(\frac{a}{g}\right) dt \times \left(\frac{dv}{dt}\right), \text{ or, } \Delta h = \left(\frac{a}{g}\right)\Delta V$$

Calling this additional change in velocity (of the end of the column), $\Delta_2 V$, we may write:

$$\Delta h = \left(\frac{a}{g}\right)\Delta_2 V$$

but,

$$V_{n-1} - V_n = \Delta_1 V + \Delta_2 V, \text{ and, } \Delta h = h_n - h_{n-1}$$

Hence,

$$2h_{n-1} + h_n - h_{n-1} = h_n + h_{n-1} = \left(\frac{a}{g}\right)(V_{n-1} - V_n)$$

Mr. Warren

MINTON M. WARREN,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper goes into the mathematical theory of ordinary water-hammer more thoroughly than any previous discussion of the question, and is certainly a valuable addition to the literature on this subject. The curves presented by the author show the differences in the various formulas with great clearness.

The writer, however, does not believe that Mr. Gibson's method and formulas are practical for ordinary use for three reasons:

First.—In order to use the formula, a large amount of very tedious figuring must be done, and unless it is very carefully checked by logarithms, a very small arithmetical mistake may make a large error in the result.

Second.—The formulas have not been confirmed by experiments and are based on certain assumptions the accuracy of which are open to question.

Third.—In most cases, simpler formulas will give as accurate results as the data warrant.

It is almost impossible to get away from the idea that the more calculations and exact mathematical methods used in obtaining a result in engineering, the more accurate the result will be, regardless of the assumptions on which the calculation is based. It is the same false accuracy that leads engineers to submit cost estimates figured down to odd cents on projects running into millions.

The writer does not believe that the many assumptions needed in deducing any formula for slow-closing gates warrant the elaborate methods used by Mr. Gibson, until these methods and formulas are backed up by careful and extensive experimental data, as were the formulas of Professor Joukovsky.

In arriving at his results, Mr. Gibson assumes that the area of the gate is closed at a uniform rate, whereas the writer, in deducing his simple formula,

$$h = \frac{L\,V}{g\left(T - \frac{L}{a}\right)}$$

assumed that the gate moved in such a way as to cause the pressure to rise at a constant rate. In any given case, neither assumption is strictly true, and experiment alone will show which is nearer the average gate motion.

Referring to Fig. 9, it is seen that, according to the author's calculations, such a gate motion, giving a constant rise of pressure, is not far from a straight line, and probably as near the truth in ordinary

* Boston, Mass.

Mr. Warren. gates as his assumption (uniform reduction of area), which makes the mathematics very much more difficult.

There is great need for a series of careful experiments on slow-closing gates, and it is to be hoped that Mr. Gibson, or some other member of the Society, will undertake this work. Data from such tests properly used would be of more service to the Profession than the most perfect theoretical formulas which have never been tested. Until this is done, engineers will have to base design on the meager practical data available, guiding their judgment by unproved formulas, and the writer has found that the simple formula given above comes nearer the pressures he has observed in practice than other formulas, although, like the others, it is based on certain assumptions which are not strictly true.

In recent tests by one of the best-known manufacturers of water-wheel governors, the author's formula has given results so closely in accord with the tests, that it has been adopted for general use. As this company has many chances to obtain practical operating values of water-hammer, its conclusions are certainly of interest, in default of other experimental data.

In regard to Mr. Halmos' statement that this formula can never give true results, even accidentally, it should be noted that, under certain conditions, it gives the same results as the formula which he uses (Allievi's), and, in some of the tests mentioned in the previous paragraph, has given results within the probable error of the pressure gauges used.

In one point, Mr. Gibson's formula gives what appears to the writer unreasonable results. This is illustrated in Fig. 4, where the curve rises very sharply between heads of from 10 to 100 ft. Alliévi's curve rises even more sharply and can be easily proved wrong, as it reaches values of h which are far above the maximum value possibly reached in instantaneous closing.

Mr. Gibson's curve stops at that value, but the writer does not believe that experiments confirm the large increase in water-hammer for low heads over that for high heads. His assumption that the waves are perfectly reflected from the slowly closing gate is also open to question, and was not proved in Joukovsky's experiments.

It is not intended to imply that Mr. Gibson's formulas may not prove to be as accurate or even more accurate than the others, but, in the absence of any experimental proof, their added complexity does not seem to be warranted, in view of the approximations and assumptions on which they are based.

Mr. nderson. J. T. Noble Anderson,* M. Am. Soc. C. E. (by letter).—On several occasions, the writer has investigated this question, or a cognate phe-

* Narbethong, Victoria, Australia.

nomenon, with pumps—centrifugal and others—through long pipes. In one case, there were two pipes with a common penstock—each of wood, 3 ft. in diameter, and 2 300 feet long. In every case, he has been greatly impressed by the extreme complexity of the subject. Mr. Anderson.

With the help of Professor Joukovsky's theory and some "rule-of-thumb" guesses, gained from a general experience, these pipe lines and the values concerned can now be calculated to as close an economic margin as most other engineering problems. At the same time, the result of experience shows so many baffling discrepancies, that the experimental results, which the author hopes to obtain "in the not too distant future", will be waited for eagerly.

The care he has taken to collect and co-ordinate the formulas, is an earnest that he may be relied on to record fairly and fully not only his results, but all the extraneous facts and happenings which may, however remotely, bear on the many irregularities between actual records and what the formulas anticipate. In particular, it might help if two slightly disturbing factors were recorded in each test, that is: (1) the temperature of the water, and (2) the proportion of free air present in the water.

The second factor will probably be quite immaterial, but, in a case where the substance pumped through a centrifugal was sewage with a specific gravity of 64 lb. per cu. ft., the writer found that the gases contained had a very disturbing influence.

In some recent investigations, the writer used a steam indicator with springs specially chosen for the anticipated pressure. Some indication of the author's apparatus will no doubt be given.

Ford Kurtz,* Esq.† (by letter).—Mr. Gibson has made a valuable addition to the too few and scanty English treatises on the mathematical theory of water-hammer. His treatment of the subject, however, is chiefly of value in obtaining, without the use of differential equations and from physical laws the import of which is readily grasped, formulas which give the same practical results as the much simpler and less cumbersome equations of Lorenzo Alliévi, first published in Rome, in 1903. These equations must not be confused with the confessedly approximate and inadequate formula designated by Mr. Gibson as the "Alliévi formula", and on which Mr. R. D. Johnson has apparently founded a pressure-time equation. The equations referred to are mathematically rigid formulas which take into account not only the effect of net head but also the compressibility of the water and the extensibility of the pipe and which, as far as the writer knows, have never before been published in English. In 1911, the writer prepared, for his own use, a translation of a German translation of Mr. Alliévi's Mr. Kurtz.

* New York City.

† Now M. Am. Soc. C. E.

Mr. Kurtz.

work, and found the latter's treatment so remarkably comprehensive and thorough that he has used it ever since in all water-hammer problems. The German translation* can be found in the Engineering Societies Library.

In presenting the exact formulas of Alliévi, the following nomenclature will be added to that of Mr. Gibson's paper:

$R_0 = \frac{a V_0}{g}$ = excess, or water-hammer, head due to instantaneous complete closure of gate.

$m = \frac{a V_0}{g H_0}$ = ratio of instantaneous water-hammer head to net head.

$F(t)$ and $f(t)$, or simply F and f = certain functions of time, t.

$z = \frac{H_0 + h_t}{H_0}$ = ratio of total variable head to net head.

$\phi(t)$ = gate-opening at time, t, as a ratio of maximum gate-opening.

The exact formulas of Alliévi contain a term which makes it possible to determine the pressure at any point of the pipe line at any moment, but the writer is presenting only the simple form for determining pressure at the outlet, or discharge section just up stream from the gate, as that is the problem investigated by Mr. Gibson. It is also assumed that the pipe line is of uniform thickness and diameter throughout its length. Then, during the period,

$$0 < t \leq \frac{2L}{a},$$

$$z = 1 + \frac{F(t)}{H_0},$$

where

$$\frac{F(t)}{H_0} = m - \frac{m^2}{2}[\phi(t)]^2 - m\,\phi(t)\sqrt{1 + m - \frac{m^2}{4}[\phi(t)]^2}$$

and, during the period,

$$\frac{2L}{a} < t \leq T.$$

$$z = 1 - \frac{F(t)}{H_0} - \frac{F\left(t - \frac{2L}{a}\right)}{H_0} = 1 + \frac{F(t)}{H_0} - \frac{f(t)}{H_0},$$

* "Allgemeine Theorie über die veränderliche Bewegung des Wassers in Leitungen," von Lorenzo Alliévi, 1909.

Mr. Kurtz.

where

$$\frac{F(t)}{H_0} = m - \frac{f(t)}{H_0} + \frac{m^2}{2}[\phi(t)]^2 - m\,\phi(t)\sqrt{1 + m - \frac{2f(t)}{H_0} + \frac{m^2}{4}[\phi(t)]^2}$$

For the linear law of gate movement, $\phi(t) = 1 - \frac{t}{T}$, as already stated by Mr. Gibson.

These formulas applied to Mr. Gibson's first example give the following equations:

For $0 < t \leqq 0.35$ sec.,

$$\frac{F(t)}{H_0} = 10.35008 + 53.56185\,\phi^2 - 10.35008\sqrt{11.35008\,\phi^2 + 26.78092\,\phi^4}$$

and, for $0.35 < t \leqq 6.0$ sec.,

$$\frac{F(t)}{H_0} = 10.35008 - \frac{f(t)}{H_0} + 53.56185\,\phi^2 - 10.35008\sqrt{\left(11.35008 - \frac{2f(t)}{H_0}\right)\phi^2 + 26.78092\,\phi^4}$$

As already noted by Mr. Gibson, the values of $\frac{F(t)}{H_0}$ and $\frac{f(t)}{H_0}$ are so small compared with some of the individual terms of the equations, that it is necessary to use logarithms in solving the equations.

Using the exact Alliévi equations as given previously, Table 6 has been prepared, showing the rise of pressure and also the differences between the values of the rise obtained by Alliévi equations and those obtained by Mr. Gibson's equations. The maximum divergence of less than one-half of 1% shows the remarkable agreement of the two methods. It is apparent, however, that the simplicity of the solution by the Alliévi formulas, without giving heed to magnitude and direction of waves in the computations, makes it far superior to that of Mr. Gibson as a working method.

The exact formulas of Alliévi also furnish equations similar to those given previously for the case of the opening of a valve at the lower end of a pipe line, either from fully closed position, or from some initial partial opening. As already stated, in their complete form they also give the pressure at any time for any point along the pipe line, thus covering the matter mentioned by Mr. Gibson as to be discussed by O. V. Kruse, Assoc. M. Am. Soc. C. E. Of course, there are also exact equations for the velocity at any time for any point along the pipe line.

Mr. Kurtz.

The partial differential equations for the general motion of water in pipes are based on the fundamental differential formulas for the motion of water in general. Unfortunately, these partial differential equations, four in number, cannot be integrated (not even by approximate arithmetic integration so far as the writer knows) without making the following simplifying approximations:

1. Velocity in direction of axis of pipe considered uniform over any chosen section of the stream.

2. Skin friction and viscosity neglected.

3. Velocities at right angles to the axis of the pipe, due to expansion or contraction of the pipe by changes in pressure, neglected.

4. Pressure considered uniform over any chosen section of the stream.

5. Assumed that the pipe consists of individual circular elements independent of each other, which are freely extensible.

6. Assumed that the ratio of velocity of water in the pipe to the velocity of propagation of pressure changes is small enough, compared with unity, so that its addition thereto or subtraction therefrom can be neglected in every case at every instant.

By making these approximations, we obtain the so-called exact formulas of Alliévi.

Mr. Gibson's method of taking account of skin friction (which is only approximate, as has been pointed out by William P. Creager, M. Am. Soc. C. E.) could easily be applied to the exact Alliévi formulas by changing the factor m so as to have it correspond at all times to $(H_0 + h_f)$ instead of to H_0. The exact Alliévi formulas can be applied so as to take account of varying diameters and thicknesses of pipe in the same line, but they soon lead to so much complication that they become impracticable. In such cases, the writer uses the formulas as already given for a pipe of uniform diameter and thickness, but gives to m the value, $\dfrac{Q_0 \Sigma \dfrac{L}{A}}{g\, H_0 \Sigma \dfrac{L}{a}}$, where Q_0 equals the flow of water in a pipe at full gate-opening, L equals the length of any section of pipe, A, its cross-sectional area, and a, its individual value of the velocity of propagation of pressure changes. Also, the factor, $\dfrac{2\,L}{a}$, must everywhere be changed to read $2 \Sigma \dfrac{L}{a}$. This is confessedly an approximation made without mathematical proof, but it is probably exact enough for practical purposes in the majority of problems.

TABLE 6.

Mr. Kurtz.

Interval in terms of $\frac{2L}{a}$	$\phi(t)$	$\frac{F(t)}{H_0}$	$\frac{f(t)}{H_0} = \frac{F\left(t-\frac{2L}{a}\right)}{H_0}$	$z-1$	h_t, Allévi	h_t, Gibson	Percentage of divergence.
0	1.00000	0.00000	.....	0.00000	0.00	0.00	.....
¼	0.95833	0.07338	.....	0.07338	12.10	12.12	0.16
½	0.91667	0.15476	.....	0.15476	25.53	25.53	0.00
¾	0.875	0.24503	.....	0.24503	40.43	40.41	0.05
1	0.83333	0.34550	0.00000	0.34550	57.01	56.96	0.09
¼	0.79167	0.49780	0.07338	0.42442	70.03	69.94	0.13
½	0.75	0.66286	0.15476	0.50810	83.84	83.72	0.14
¾	0.70833	0.84160	0.24503	0.59657	98.43	98.31	0.12
2	0.66667	1.03529	0.34550	0.68979	113.82	113.63	0.17
¼	0.625	1.26152	0.49780	0.76372	126.01	125.78	0.18
½	0.58333	1.50125	0.66286	0.83839	138.33	138.05	0.20
¾	0.54167	1.75454	0.84160	0.91294	150.64	150.32	0.21
3	0.5	2.02152	1.03529	0.98623	162.73	162.42	0.19
¼	0.45833	2.30592	1.26152	1.04440	173.33	171.97	0.21
½	0.41667	2.60055	1.50125	1.09930	181.38	181.10	0.15
¾	0.375	2.90456	1.75454	1.15002	189.75	189.43	0.17
4	0.33333	3.21686	2.02152	1.19534	197.23	196.77	0.23
¼	0.29167	3.53609	2.30592	1.23017	202.98	202.49	0.24
½	0.25	3.86010	2.60055	1.25955	207.83	207.14	0.33
¾	0.20833	4.18755	2.90456	1.28299	211.69	211.00	0.33
5	0.16667	4.51715	3.21686	1.30029	214.55	213.79	0.35
¼	0.125	4.84719	3.53609	1.31110	216.33	215.75	0.26
½	0.08333	5.17707	3.86010	1.31697	217.30	216.71	0.26
¾	0.04167	5.50630	4.18755	1.31875	217.59	216.95	0.28
6	0.00000	5.83293	4.51715	1.31578	217.10	216.57	0.23
Average percentage	.....	.....	.....	.....	.....	.....	0.20

Mr. Creager.

WILLIAM P. CREAGER,* M. AM. SOC. C. E. (by letter).—The theory of water-hammer in penstocks is one of the most intricate problems confronting engineers. Mr. Gibson has made a considerable addition to the knowledge of the subject; but exact solutions for all conditions have not yet been reached.

The author has developed rational equations for penstocks of constant diameter and negligible friction head. In cases where friction head is relatively large, and particularly where the penstock has a varying diameter, we are still very far from a practical solution.

At any instant during gate closure, the discharge through the gate is a function of the static head plus water-hammer head less friction head, all measured at the gate.

On page 725, the author makes the assumption that the friction head at the gate is proportional to the square of the velocity adjacent to the gate. This assumption appears to the writer to be only approximate, since it is well known that, at any instant during surges, the velocity is materially different at different points on the penstock.

* New York City.

Mr. Creager.

It is the writer's opinion that, during the period, $\frac{2L}{a}$, subsequent to a single small instantaneous closure, the friction head at the gate is not constant, although the velocity adjacent to the gate during that period is constant. This constantly varying friction head makes it very difficult to include this feature in the equations for water-hammer and, in all probability, the author's methods are as close as can be obtained. It would be of interest to know how much difference the inclusion of friction head makes in ordinary problems.

For a penstock with varying diameter, auxiliary waves are set up each time a water-hammer wave passes a point of change in diameter. For this condition, the author's equations would not apply. His methods would apply, but they would be exceedingly difficult, if not impossible, of practical application.

It is evident that the maximum rise of pressure is materially influenced by the characteristics of the gate-closure curve. It is well known that modern turbine governors do not provide a uniform rate of gate closure throughout the stroke. It is also possible that governors of different types have different closure characteristics. Consequently, complete data for the determination of maximum water-hammer for specific cases must include the gate-closure characteristics of the governor which is to be a part of the machinery.

Mr. Gibson.

Norman R. Gibson,* M. Am. Soc. C. E. (by letter).—In closing the discussion of his paper, the writer desires, first of all, to make amends, as far as possible, for the injustice unknowingly done to Mr. L. Allievi by the reference to his formula on the diagrams and in the text of the paper. As pointed out by Mr. Halmos, the formula designated by the writer as Allievi's is only one of several contained in his complete work. While "ignorance excuseth none", the writer can only plead, in extenuation, that he did not read Allievi's work, because he could not find an English translation of it, but was led to believe, by the frequent references to "Allievi's formula" in English literature on the subject, that the formula quoted was, in fact, the one and only equation derived by Allievi. The writer was the more completely led astray by a lengthy reference to this formula in a paper published in the *Transactions* of the Society, from which it could not but be inferred that the author had read the work of Allievi in the original language. It is inconceivable how any one, having read it, could so overlook the most important part of Allievi's work.

The writer is indebted to Mr. Halmos and Mr. Kurtz for having brought to light some further formulas of Allievi, as published in the German and Italian languages. Mr. Halmos states that the writer's solution agrees in every particular with that of Allievi, and Mr. Kurtz finds a close agreement in an arithmetical example.

* Niagara Falls, N. Y.

The identity of the results obtained by the Allievi equations with those derived from the writer's may be proved as follows: Mr. Gibson.

Considering first Equation (3) presented by Mr. Halmos on page 750, it should be noted that Mr. Halmos had the writer's attention drawn to a mistake in his definition of $\psi\ (t)$. This symbol stands for the ratio of the instantaneous area of discharge to the area of the pipe (not the area of the gate-opening at the time, $t = 0$).

The two nomenclatures are as follows:

Halmos.	Gibson.
$\psi\ (t) = \left(1 - \frac{t_n}{T}\right) \frac{B_0}{\sqrt{2\ g}}$	$= \sqrt{\frac{S_{t_n}\ g}{2\ a^2}}$
$y_0 = H_0$	
$y = H_0 + h_{t_n}$	
$H = H_0 + \frac{a\ V_0}{g}$	
$c_0 = V_0$	
$C = V$	

On page 750, Mr. Halmos states that $F\ (t)$, found in the first interval, should be substituted for (f) in Equation (3), in order to obtain the value of y for every subdivision of the second interval.

Using the subscript n of the writer's notation, the relation between $F\ (t)$ and (f), may be expressed by the equation:

$$f\ (t_n) = F\ (t_{n-1})$$

Again, on page 750, Mr. Halmos gives:

$$F\ (t) = \frac{a}{g}\ (c_0 - C) - f$$

Substituting the subscript, n, and the writer's notation for c_0 and C,

$$F\ (t_n) + f\ (t_n) = \frac{a}{g}\ (V_0 - V_{t_n})$$

$$\therefore\ 2\ \frac{a}{g}\ (V_0 - V_{t_n}) = 2\ \{F\ (t_n) + F\ (t_{n-1})\} \quad \ldots\ldots\ldots\ (1)$$

and, similarly,

$$2\ \frac{a}{g}\ (V_0 - V_{t_{n-1}}) = 2\ \{F\ (t_{n-1}) + F\ (t_{n-2})\} \quad \ldots\ldots\ldots\ (2)$$

Subtracting Equation (1) from Equation (2):

$$2\ \frac{a}{g}\ (V_{t_n} - V_{t_{n-1}}) = 2\ \{F\ (t_{n-2}) - F\ (t_n)\}$$

If each term in the writer's general expression for R is now expanded in terms of V, then:

$$R_{t_n} = R_0 + 2\ \frac{a}{g}\ \{(V_{t_n} - V_{t_{n-1}}) + (V_{t_{n-2}} - V_{t_{n-3}}) + \ldots\ldots\ldots\ldots\}$$

Mr. Gibson.

and, by analogy:

$$R_{t_n} = R_0 + 2\left[\{F(t_{n-2}) - F(t_n)\} + \{F(t_{n-4}) - F(t_{n-2})\} + \ldots\ldots\right]$$

and, by cancellation:

$$R_{t_n} = R_0 - 2F(t_n)$$

and

$$R_{t_{n-1}} = R_0 - 2f(t_n)$$

By substitution of the above values and by reference to the relation of terms in the paper, Allievi's Equation (3), as given by Mr. Halmos on page 750, may be written for any interval, n:

$$(H_0 + h_{t_n})^2 - 2(H_0 + h_{t_n})\left(H_0 + R_{t_{n-1}} + \frac{S_{t_n}}{2}\right) + \{H_0 + R_{t_{n-1}}\}^2 = 0$$

and by solving this quadratic:

$$h_{t_n} = \frac{1}{2}\{(S_{t_n} + 2R_{t_{n-1}}) \pm \sqrt{St_n(St_n + 4R_{t_{n-1}} + 4H_0)}\} \ldots (3)$$

which is the writer's general expression, similar to Equation (10) on page 724, for h at any time in any interval.

Mr. Kurtz has presented the general Allievi formula in somewhat different form from that used by Mr. Halmos, in that the values of V and h at the interval points, as contained in the writer's values of R, are eliminated, and the labor of computation is thereby reduced. The writer's equations, however, require no more heed to be given to "magnitude and direction of waves in the computations" than do the Allievi equations. Mr. Kurtz's remark in this connection, on page 767, probably refers to the explanation of the work of arithmetic integration which preceded the writer's derivation of his formulas. Mr. Kurtz's Table 6 of arithmetic errors indicates merely the difference in the results that might be obtained by two computers. The results given by the Allievi formulas are identical with those of the writer.

The elimination from the writer's equations of V and h at the interval points may be accomplished in the following manner: Referring to the equations on page 724, the expression for R_{t_1} may be equated to that part of the equation remaining after R_{t_1} has been substituted in place of $\frac{a}{g}V_1 + h_1 - C_{t_1}$.

Thus, in general,

$$R_{t_{n-1}} = \frac{a}{g}\left(1 - \frac{t_n}{T}\right)B_0\sqrt{H_0 + h_{t_n}} + h_{t_n} = \frac{a}{g}V_{t_n} + h_{t_n}$$

$$= \frac{a}{g}V_n + \frac{a}{g}(V_n - V_{t_n}) + h_{n-1} - C_{t_{n-1}} = \frac{a}{g}V_{t_{n-1}} - h_{t_{n-1}}$$

$$= \frac{a}{g}\left(1 - \frac{t_{n-1}}{T}\right)B_0\sqrt{H_0 + h_{t_{n-1}}} - h_{t_{n-1}}$$

$$\sqrt{S_{t_{n-1}}(H_0 - h_{t_{n-1}})} - h_{t_{n-1}} \ldots\ldots\ldots\ldots (4)$$

If this value of R_{n-1} is substituted in the writer's equations in place of the value given in the nomenclature on page 722, the labor of solving problems by the formulas will be lessened. Mr. Gibson.

Mr. Johnson's discussion contains considerable new matter which will be found most helpful in solving problems of the kind under consideration. His remarkably simple and concise presentation of the fundamental principles of this subject and his derivation of the expression for $\frac{h_n}{H_0}$, the ratio at any time of the rise of pressure to the net head, are particularly noteworthy and important. His chart of limitations, shown in Fig. 12, makes it possible to see at a glance where it is safe to apply the general logarithmic curve given by Allievi's Formula (4), which is Mr. Johnson's Equation (7), and also to see where the maximum pressure rise occurs at the end of the first interval. As pointed out by Mr. Johnson, his Equation (7) covers a wide range of conditions, and it will be found convenient to use this equation wherever it applies. In Fig. 13, Mr. Johnson has indicated how an exceedingly useful chart may be prepared by resorting to the expedient of solving for the ratio, $\frac{h_{max.}}{H_0}$, instead of for $h_{max.}$ directly. Such a chart may be made to show graphically a range of results as wide as desired and, as stated in the discussion, each point on the chart covers an infinite number of cases.

Mr. Creager's desire to know more about the application of water-hammer formulas to pipes of varying diameter will be appreciated by every one who studies this subject. Needless to say the problem is a complex one, but it seems certain that the fundamental relations that have been developed herein can be applied not only to this but to any water-hammer problem. The point brought out by Mr. Creager that the friction head at the gate is not constant during the period of $\frac{L}{a}$ is worthy of consideration, but the writer would suggest that the change in friction head, being a change in pressure, cannot traverse the length of the pipe at a rate faster than the velocity, a. In any event, even though the friction head does vary during the interval periods, it would seem that the net result throughout the closing time would be that the friction and velocity heads are recovered proportionately to the rate at which the velocity of flow is destroyed.

On page 769, Mr. Creager misquotes the assumption made on page 725 of the paper. The friction head at the gate was assumed to be proportional to the square of the velocity of flow, not the velocity "adjacent to the gate."

During the past year many experiments on pressure rise have been made by the writer with specially designed apparatus. Most of the ex-

Mr. Gibson.

periments, however, were made in connection with his new process for the measurement of the flow of fluids in closed conduits, and as this work continued until late autumn, it has been necessary to postpone until later the particular experiments that have been proposed to show the relation between gate motion and pressure rise covered by the formulas. As soon as possible the results of these experiments will be made available for publication, and in assembling the data Mr. Anderson's request for full particulars will be remembered.

In reply to the criticism of Mr. Warren, it may be observed at once that the vital importance of including the initial static head among the factors that determine the maximum rise of pressure, cannot be considered in the same class as "odd cents in million dollar estimates". The large increase in water-hammer for low heads over that for high heads when the gate motion is uniform, is caused by the influence of the initial head.

Mr. Warren cannot obscure the points at issue by clouding the discussion with conjectures as to whether uniform gate motion or the gate motion he has assumed, but not defined, "is nearer the average gate motion". The vagaries of gate motions are not under consideration, as is clearly stated on page 710. The problem considered is the rise of pressure following any given gate closure, and, for this purpose, in deriving the formulas, uniform gate motion has been assumed. Diagrams of non-uniform gate motion were shown and attention was drawn to the effect of such non-uniform motion on the shape of the pressure-time diagram. In all practical cases, actual conditions will be determined and, as Mr. Johnson says, "the element of judgment must enter as usual". Mr. Warren's statement that his formula is based on a certain prescribed shape of the pressure-time diagram which may be produced by a special, though undescribed, performance of the gate, condemns his formula for general use, because it is impossible to adapt it to any particular gate motion. Indeed, what he says of his own arbitrary diagram is equally true of any form whatever which gives a value of the maximum pressure rise greater than the mean effective pressure rise and which encloses a proper impulse area.

For uniform closure, Mr. Warren's formula always leads to errors on the wrong side, that is, it gives less than the true value. In the region, pointed out in the paper, where it is approximately correct, and when the first interval is a small proportion of the closing time, as it frequently is in ordinary cases, this formula gives a result close to the value of the mean rise of pressure which can be determined by the simple expression, $\frac{L\,V}{g\,T}$.

Mr. Warren questions the assumption that pressure waves are perfectly reflected from the slowly closing gate. The question implies a

misconception of the nature of the waves themselves. They are not of a substance that rebounds like a rubber ball, but are conditions of the water column at various times at different points along the pipe line. When a wave is started by a movement of the gate, the water is put into a state of super-normal pressure and it remains in this condition at the gate until the wave has traversed the length of the pipe to the origin and back again. When the wave has returned to the gate, the state of super-normal pressure there is converted to one of sub-normal pressure and the water there remains in this condition until the wave has again traversed the distance to the origin and back, whereupon the condition at the gate again changes to a state of super-normal pressure, and so on. At points along the pipe the changes of condition occur in the same sequence, except that a state of normal pressure exists for a time between the periods of super-normal and sub-normal pressures. When the gate is gradually closing, the algebraic sum of the innumerable infinitesimal waves (super-normal pressure being considered positive and sub-normal pressure negative) fixes the pressure that exists at the gate at any time during the closure. Mr. Gibson.

The authority for the assumption that the flow of water in the pipe does not affect the wave is to be found on page 709 in the following quotation from Miss Simin's translation of Joukovsky's work:

"If the water column continues flowing, such flow exerts no noticeable influence upon the shock pressure. In a pipe from which water is flowing, the pressure wave is reflected from the open end of the pipe, in the same way as from a reservoir with constant pressure."

Finally, it may be stated definitely that, as an empirical rule for convenient use, Mr. Warren's formula is quite untrustworthy, and as an equation showing the relation between velocity destroyed and rise of pressure, it is fundamentally and inherently wrong.

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Paper No. 1440

ADDRESS AT THE ANNUAL CONVENTION IN ST. PAUL AND MINNEAPOLIS, MINN., JUNE 17TH, 1919.

By Fayette S. Curtis, President, Am. Soc. C. E.

For the last two years the Annual Convention of the Society has been omitted on account of the war; therefore, the Presidential Address has been delivered at the Annual Meeting in January. The fact that we are again in a position to resume our Convention meetings is, I am sure, a source of unbounded gratification to all.

In this address I ask pardon if I digress somewhat from the trend of former addresses and speak of the fundamental principles of the engineer, his duties, what he has accomplished and can accomplish in the future, the application required to make work successful, and the distinction that may be gained through uprightness and honest labor.

It is quite probable that some of you have often been asked the question: "Do you recommend Civil Engineering as a profession?" and I assume that generally an affirmative answer has been given. The civil engineer, when thoroughly fitted for his profession, immediately becomes useful to society and the world of materiality. He is prompted by the wants of society, and is constantly originating other wants by the improvements he introduces. He is also creative, and his profession is not mortgaged to power or influence, but stands firmly on its own foundation—encouraged by world-wide influences.

His duty consists in designing, arranging, and building structures or machines which require the immediate superintendence of those who are acquainted with not only the principles but the practice of con-

struction; for engineering is a branch of knowledge that takes its place both among the Sciences and the Arts.

Therefore, to be eminent in his profession, or any branch of it, he should be able, not only to design and plan, but to supervise construction. Those who have only the knowledge of how to design and plan, and are unable to execute, become confused in meeting the many contingencies which arise.

The profession, embracing as it does almost every kind of construction, requires a very extensive and general acquaintance with other sciences, in order that the engineer may be qualified to accomplish successfully the various works on which he may be engaged, and to overcome those difficulties which frequently arise unexpectedly in the progress of his work, and, but for that knowledge and talent, threaten its final success.

Thomas Telford, in his "Descriptive Narrative of his Professional Labours", has said:

"How can a man give judicious directions unless he possesses personal knowledge of the details requisite to effect his ultimate purpose in the best and cheapest manner? It has happened to me more than once, when taking opportunities of being useful to a young man of merit, that I have experienced opposition in taking him from his books and drawings, and placing a mallet, chisel or trowel in his hand, till, rendered confident by the solid knowledge which experience only can bestow, he was qualified to insist on the due performance of workmanship, and to judge of merit in the lower as well as the higher departments of a profession in which no kind or degree of practical knowledge is superfluous. For this reason I ever congratulate myself upon the circumstances which compelled me to begin by working with my own hands, and thus to acquire early experience of the habits and feelings of workmen; it being equally important to the Civil Engineer, as to Naval or Military Commanders, to have passed through all the grades of their profession."

It is no doubt true that many men have gained prominence in their profession by individual effort—practicing and studying at the same time, with a view to acquiring a full knowledge of all necessary scientific principles. Students are required to spend a certain time in acquiring technical knowledge, but must not then think they are

engineers in the broad meaning of the term, capable of constructing and being responsible for engineering work, as that is a false idea.

No one can doubt that the addition of the natural sciences to the old-time arts course has greatly enriched it, and the sciences are now undoubtedly the most important part of a liberal education; but there is another side, which we cannot ignore, and that is, that after all, the great object of a good education is not so much the accumulation of knowledge as the development of mental power in the individual.

The tendency toward scientific and practical education is no new thing. Men have come to see that the first mental training can be made more effective by solving physical problems than by studying the dead languages, because as much true value can be obtained in the study of Nature as in the study of man's conception of Nature. First knowledge is more satisfactory to the best of minds.

The profession has become so diversified that after having secured a good fundamental training it would be worth serious consideration to take up a single branch of engineering practice, as it can hardly be expected that the civil engineer will become proficient in every branch. He would be better able to give his best endeavors to attaining the highest proficiency in some particular line, and, at the same time, grasp or understand in a general way the subjects which may relate to the several kinds of works, as the whole subject is so broad that it would be vain to attempt to understand them all in all their details.

Ambition to obtain as great a knowledge as possible is to be admired and encouraged, but to attain the highest standard in one branch is more commendable than to attempt to reach special eminence in all, as the student will be apt to find himself oppressed with a weight which he is not capable of sustaining, and this will be very likely to result in utter failure.

The professional man should not make the mistake of thinking that he can reach the top in an unconcerned and idle manner, believing that by some easy process he will reach and hold it. The structure on which he stands must be constructed by piece and parcel, every member thoroughly tested by a gradual and difficult process; he must experience all the difficulties, trials, and troubles to be met on the way, in order to support and sustain him at the higher elevation to which he aspires. To such the doors to success are open. Work seeks the best hands, as naturally as water runs down hill, but seldom seeks one whose only recommendation is that he needs it. If he is not willing

to go slowly through the maturing process of time, through patient work, by steady application step by step preparing himself for the work ahead, which has made men prominent and efficient in their profession, then, alas, the result is doubtful. It is like the student who prepares his education for future advancement. He goes from grade to grade until he is master of the science on which he has to build, taking no upward step until the last has been tested and found not wanting in any characteristic which will sustain him and serve him to build on for the future.

New problems are all the time arising. The tremendous growth of our complex industrialism means the facing of new conditions, that we may overcome new difficulties and enjoy new benefits. Therefore, there is only work to face, and it should be met with a spirit characteristic of the creator of science, and thereby much may be accomplished.

There is no success without hard and earnest labor. Never balk at a few obstacles, but hang on with all your might; this, in the end, will give success. It is true that many experiments will have to be made, but there can be no doubt of ultimate success if our aim is a determination to succeed and we do not become discouraged.

As we look back, even over a short time, how often have we said to ourselves, after completing some important structure, "This should last for many years—we have reached the limit;" but within a comparatively brief period have we not seen that structure replaced by another, larger and more expensive, which has been demanded by the growth of business? We have not stopped building, nor have we come to the time when we have built sufficiently for the future, and we never will, as long as the richness of this country is undeveloped and its physical and financial growth continues. To stop this great advancement, which the people continue to make, would be death. Then, does it look as though there was not a future for the engineer who has integrity and perseverance, as he is and must be the practical foundation or basis for all which we are to build, in order to accomplish a higher and nobler position in society?

The work of the engineer is the foundation on which rests the growth of this great Republic, not only physically but financially. He is one of the prime factors, and could not by any possibility be dispensed with. We are not an organization which threatens to pull

down and wreck or destroy what we have built, but an educational organization, and hope to get the necessary recognition of our rights through education. No doubt the engineer has often had his line of procedure curtailed by some power having authority to suspend or discharge at will, at the pleasure of some controlling power, ignorant, and with no understanding of the conditions or the difficulties the engineer may have to meet, and without the least knowledge of engineering subjects.

Civil engineering is ages old, but has always shown a steady progression, possibly not as to more substantial work, but certainly in the facilities provided in machinery to do its work. As to its beginning, I will not undertake to say, but will only refer back to those times of which we may have knowledge, and ask if the great work done is not a credit to the scientific ability of this profession? But what is the future to be? Does it depend on those who are coming into this Society —educated, experienced—to show to the world that they can and will continue to build and improve the financial and social conditions? For a determination we must wait on time.

Is the engineer being benefited by his connection with this Society? It has been said that the Society is becoming less democratic and more conventional, or, in other words, is not taking sufficient interest in, or having proper regard for, its members, and, therefore, they are without the opportunities or advantages which they might gain in local societies. This Society has grown to such an extent that contact personally with all its members is not possible, but the interest and devotion and brotherly love of its members are just as great as ever. The information in the different papers by the most prominent engineers of the country, presented and distributed, is of the greatest importance to the members of the Society. As the Society grows, this information increases, and the papers are more numerous and contain information and knowledge which cannot be obtained except through the parent society. If not, why are so many engineers anxious to affiliate themselves with this Society? They know it gives them prestige the world over, and sets a standard which has been passed upon and approved by the highest of professional men. Our Constitution requires that standard to be higher than that of any of the local societies.

The Society has not only grown in numbers, the increase within the last ten years being 85%, but has increased in influence in the estimation of the public and the world at large; our members are looked up to as the leaders in the Engineering Profession. Then, who can say that it, as an engineering society, has depreciated, gone back, or fallen off one iota from its former standing? On the contrary, it has grown steadily stronger and stronger. Its members have become of such strength, in the material and financial upbuilding of the Government, that they are indispensable.

Therefore, we must not permit ourselves to forget the obligation we owe to the founders of this Society, the men who gave their time and talent in creating an organization which has proved to be of untold benefit to the Engineering Profession. How, then, are we to repay them, if not by giving to the Society our support and assistance, promoting good fellowship among its members, encouraging its officers, and giving a helping hand to the future engineer who comes into the Society, passes up through the different grades of experience, and is seeking information? Let us all do something in the interest of the Society, and thereby broaden its influence and add to its high standards of proficiency.

We certainly cannot help by creating dissensions among the members, or becoming dissatisfied with the earnest labors of those whom we select to bear its responsibilities, or in getting the idea that we are connected with this Society wholly for what we can get out of it. No matter how much we give, we can never repay the Society for the benefits we have received.

The engineer has designed and built many a monument which will endure for ages as a testimonial to his proficiency and skill. We need not go back many years to find members who have won the highest engineering distinction and whose names are ineffaceably engraved on the enduring pages of history. We know who they are and what they have accomplished. Even before this Society was organized, we find the names of engineers who achieved everlasting remembrance through their exceptional ability. Therefore, I would say to engineers, but more particularly to the younger members of the profession, that there is a future for them which, if taken advantage of, will ensure the highest honors possible to be obtained.

By him who is in the prime of life and in the possession of health, strength, and knowledge of the arts and sciences, much may be accomplished. Therefore, never give up. Think, work and suffer, that in this life you may gain something which will place your name upon the highest pinnacle of fame.

Many of our members have been engaged in the late war, and their work, in the construction of yards, docks, cantonments, and the reconstruction and building of bridges, roads and equipment, both in this country and on the other side of the Atlantic, thus making it possible to take care of millions of troops, was the most important factor in bringing the war to a successful termination. All of this has been done in such short time as to bring out comment and praise from the whole world. Why should not the engineer have his full due and credit for the ability he has shown, not only at critical moments during the war, but in the accomplishment of things at other times?

We are living in an age in which we possess all the information as to what has been done for many years, all of which we ought to profit by. The conditions change, the opportunities grow greater. This world is growing in its necessities almost beyond the imagination of the present generation. The country is being developed so that every State in the Union is increasing in wealth and prosperity. Greater facilities are needed to accommodate its growth. And yet some say there is no future!

The work of other engineering societies or organizations should also be mentioned. They have all done their part in building and developing the great industries of the world. All are working toward the same purpose, creating, building, and improving the facilities on which the prosperity of the country depends. This is illustrated by the establishment of Engineering Council, composed at present of representatives of the American Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and the American Society for Testing Materials, for the purpose of giving more strength to the profession and obtaining greater recognition for the engineering arts and sciences.

Why should we not go farther, and even take our position in Governmental affairs? Is it the political cloud that casts a shadow over us, so that our practical ability does not get the proper recogni-

tion in Government affairs; or is it our modest, indifferent way of being satisfied with our present condition? We should rise to the occasion, demand our rights, and a proper recognition by the people generally, with the idea that we are not only to advance the engineering arts and sciences but to fill the positions under this General Government which require the knowledge of how to govern, each according to his ability.

The Armistice having been signed, it is incumbent on us, as founders of prosperity, with the sciences and the arts, to become one of the greatest upbuilding factors in the future, to help reconstruct that which has been torn down and destroyed by the ravages of war, and to promote new ideas for the world's future prosperity.

Emerging from the greatest war the world has ever known, not only in the number of men engaged and the use of the most deadly weapons, but in the destruction of property, what does this mean for the future? The employment of many engineers to reconstruct, and to rebuild in an improved form.

In the future the names of many of you engineers will be enrolled among those who have attained high public standing through great achievements. Many in our profession have thought, done, and suffered much in former days. Childhood may seem to be at almost an immeasurable distance in the past, but something that we have accomplished on account of its overpowering vastness seems near. It is like a beacon light, far away but seemingly near.

It is said, "The schoolhouse of the world—human experience—however expensive, has something to teach." Shall we learn our lesson readily?

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Paper No. 1441

CONTRACTS—A COMPARISON OF "COST PLUS" WITH OTHER FORMS*

By Ernest Wilder Clarke, M. Am. Soc. C. E.

With Discussion by Messrs. F. Lavis, Missac Thompson, F. H. Frankland, J. Waldo Smith, Edward F. Hammel, Lazarus White, Arthur P. Davis, Henry H. Quimby, J. P. H. Perry, Henry B. Seaman, Frederick Glaeser, George D. Snyder, F. G. Baum, George Perrine, P. L. Reed, Walter H. Wheeler, E. G. Walker, J. A. L. Waddell, William E. Woolley, R. D. L. French, J. W. Ledoux, and Ernest Wilder Clarke.

Synopsis

1. This paper is presented in order to bring to the attention of the members of the Society the experience with the "cost plus" form of contract on Government construction work during the past two years, and to obtain from them an expression of opinion as to the advisability of a more general use of this form of contract.

2. The writer does not pretend to present any exhaustive arguments either for or against any form of contract. The paper is intended simply to form a basis for discussion, with the hope that, as a result of such discussion, some action may be taken that will be of advantage in future construction work.

3. The more or less self-evident objections to the lump-sum and item forms of contracts are mentioned, and some of the advantages

* Presented at the meeting of September 3d, 1919.

which can be obtained from the use of the "cost plus" form are also stated.

The use during the past two years of the "cost plus" form of contract on very large and complicated construction work seems to the writer to render the time opportune for a general discussion of the advantages and disadvantages of this type of contract.

In order to bring the matter before the Society, the following résumé of some of the more or less self-evident faults and virtues of the usual contract forms are presented, not with the idea of informing the members of the Society, but to establish a basis for discussion.

The usual methods of paying for work are by lump-sum, by item charges, and by cost plus a percentage. Under the first two methods, the contractor takes the engineers' or architects' specifications and estimates of the quantities, possibly checks the latter by his own computation, guesses at the interpretation which will be placed by the owner's representatives on the terms of the specifications and, from his knowledge of costs of materials and labor, makes up a bid.

In a lump-sum contract the preliminary estimate of quantities is final, as are also the original plans of foundations, details, etc. Any changes must be a matter of settlement between the owner and the contractor. The latter takes all the gamble, and if conditions or quantities turn out more favorably than was anticipated, he wins; otherwise, he loses, or is tempted to decrease the cost to himself by some method which generally means a poorer grade of work than that contemplated in the specifications. If conditions turn out much worse than anticipated by the contractor, he may forfeit whatever bond he put up and leave the owner and bondsman to settle. The owner very often desires to change the plans as the work progresses, either to cheapen the job or to add new features or improvements, and as there is no basis in the contract for payment to the contractor for such changes, a new bargain must be made. Generally, it is almost impossible to let additional work to a new contractor, so the owner is at the mercy of the original one and must submit to whatever terms he offers or forego the changes. The lump-sum contract has all the seeds of misunderstandings, disputes, lawsuits, and poor work, and not even the benefit of a pre-knowledge of ultimate cost, as there is almost invariably a long list of "extras."

In "item" contracts, the award is made on the sum of the products of the units in each item multiplied by the bid per unit. In most engineering work, the preliminary estimate of unit quantities is necessarily approximate—often the owner is unwilling on account of cost or unable for various reasons to make the investigations which would afford a basis for an exact determination. This is especially true of sub-surface structures, but even in superstructures quantities cannot always be computed accurately in advance. This is one of the great sources of disappointment to the owner in the ultimate cost of work let under this form of contract—of gambling on the part of the contractor and of loss or unfair cost to both.

The contractor and the owner may disagree on the interpretation of the data for the estimate—the contractor backs his judgment by bidding low on items which he thinks are over-estimated and high on those which are under-estimated. His total bid is lower than he would be willing to take the work for if the preliminary estimate of quantities was known to show also the final estimate, but allows a fair or better profit if the final estimate approximates that made by himself. In other words, the owner thinks he is letting the work to cost one sum, and the contractor expects to receive another and larger sum. Badly unbalanced bids are often thrown out, but a skillful bidder does not make his unbalancing so raw as to justify this action. This kind of bidding is not, *per se,* unfair or dishonest, and the ability thus to bid safely may be truly a part of the contractor's equipment, but it results in disappointment to the owner when he finds his final estimate largely in excess of that anticipated. If, on the other hand, such bidding is merely a wild guess or due to optimism on the part of the contractor, it is a gamble pure and simple which, if unsuccessful, results in loss to himself and often to the owner.

Item bids require as a basis a set of definitions. Some items are so self-evident that no confusion can possibly arise as to the meaning of the definition; others shade into other items, and it is hard to define the line of demarcation. Other items are described as containing certain materials in certain proportions, and any variation from these proportions constitutes a new item, more or less expensive. Another item is described as being a certain stock material or machine, and any variation ordered or permitted raises the question of cost to the con-

tractor. The possibility and even probability of argument and variation in cost, due to varied reading of definitions or changes in details of items, could be elaborated almost indefinitely, but enough has been said to illustrate how fruitful of bills for extra cost this part of the specification can be.

The "item" contract is better than the lump-sum in that changes in plan, due to the development of the work or the meeting of unforeseen conditions and variations in unit quantities, can be adjusted under the terms of the original contract, if the items included in that contract can be reasonably construed to apply to the new conditions; but it is still limited by the items which can be so construed and which both the contractor and owner are willing to admit are fair as to cost under these new conditions.

Applying to both these forms are several factors which are taken into account in their bids by all successful contractors and for which all owners pay indirectly. Some of these are the clauses removing responsibility from the owner for the accuracy of the preliminary data; for errors in laying out work; for refusing to accept the action of the owner's inspectors at the time of actual construction and retaining the right of rejection to the final inspection; for making the owner's engineer the final interpreter of his own specification and allowing him to supplement his drawings and specification in any way, in order to make the meaning conform to this interpretation. The contractor has to take the gamble that the original data are reasonably accurate, that the work of laying out will be carefully done, or else maintain an engineering organization to check the work—for which duplicate organization the owner will pay—that the engineers will be just, and that the original engineer will remain in charge throughout the life of the work.

Many specifications require tests of material which, if rigidly enforced, would often cause great delay and expense to the contractor or the carrying of a large stock, entailing interest charges, storage space, rehandling, and other expenses. The contractor usually expects that the purchase of material from well-known manufacturers or certificates from them will avoid the enforcement of these clauses, but if he is wise, he considers the possibility that they may be insisted on, and bids accordingly.

In short, the tendency of the entire specification is to remove responsibility from the owner and his representatives and place it on the contractor; also, all the gamble on weather, foundations, changes in labor and material market, and every other unknown or unknowable factor is carefully unloaded on him.

The contractor accepts all this, but the owner pays. The individual owner may save money, because the gamble went against the contractor on his job, but his neighbor loses, because in the long run the contractor must make money to live. The owner may lose by a contractor being too optimistic, getting a job partly done and then being unable to finish on account of high costs; re-advertising, re-letting, and delays follow, or else the bondsmen finish the work at a loss to themselves, which loss is passed on to other owners.

If a contractor makes an unexpectedly large profit, however, on Job A, he does not reduce his bid on Job B, in consequence, so that it is only the losses that are passed on to other owners and never the gains.

In "cost plus" contracts, the owner accepts all risks, all costs, and receives the benefit of all favorable conditions; each job carries its own load only, without the addition of losses on other jobs and without the percentage added by the contractor to offset possibly unfavorable conditions, ambiguous specifications, or captious owners.

The specifications and plans may be as general or as specific as the owner desires; changes can be made, difficulties met, and advantage taken by the owner of special machines, material, and methods.

The contractor furnishes the plant, organization, and expert knowledge of construction and buying; the owner supervises the work and pays the bills.

In the case of "cost plus" contracts, the contract and not the specifications is the crux of the matter from both the owner's and contractor's viewpoint, but the writing of the contract is more simple than the writing of the specifications. It is necessary first, to define what expenses are to be borne by the contractor, as not part of any one job. These may be head office expenses, salaries of chief officers whose time is spent only in part on the job, traveling expenses, except as agreed on, or the contractor's auditors, timekeepers, or material checkers. Second, to define what expenses are to be borne by the owner and are to form the basis for the fee. Such expenses would include the cost

of materials purchased through the contractor, the cost of labor of all grades, and the cost or rental of plant. Third, to define what expenses are to be borne by the owner, but which are not to be included in calculating the fee to the contractor. These might be engineering services, inspection, owner's timekeepers and auditors, and the cost of specified material which the owner elects to purchase and deliver to the contractor.

The approval of the owner would be required for all purchases of material both as to the necessity for the purchase and the price paid. The owner may reserve the right to enter the market with the contractor, purchase any material he can buy cheaper than the contractor, and deduct such cost from the total cost of the work for fee calculation. The owner would have control of the number, grade, and wages of all classes of labor, and, of course, would pay all insurance charges.

The sliding percentage with maximum fee, as used on Government work, could be applied to any contract, large or small.

Contracts could be let on the basis of fee demanded, both percentage and maximum, on the size, adaptability, and condition of the plant controlled by the bidder, and on his experience, reputation, and the size and character of his organization.

It is true that the interest of the owner to keep down the cost, and that of the contractor to receive a large fee, would clash, but that is controlled by the sliding percentage which can be arranged to give an actually larger fee for smaller total cost, and by the maximum fee which is based on the owner's estimate of total cost.

Capable, honest, efficient contractors would fare well under this form of contract; the opposite kind would soon find it impossible to secure work. Engineers of the two classes would meet the same fate as the corresponding contractors. The engineer would no longer be supposed to act as an arbiter when he is paid by one of the parties to the contract to supervise the work of the other and to secure work conforming to specifications and plans which he himself has drawn. The engineer is always biased, and no matter how hard he tries to be fair or how cordial the relation between himself and the contractor, in case of argument, the latter bears the burden of the proof. For this state of affairs the owner pays, and the successful contractor is one who has added enough to his bid, in one way or another, to make up for the

losses he suffers due, not to the work itself, but to the specifications. The writer believes that given the same men as owner, engineer, and contractor, a cheaper and better job can be secured by the owner and, in the long run, more profitable business to the contractor by the "cost plus" contract than by either the lump-sum or item bids. The owner will not be confronted by bills for extras, vexatious claims for adjustments due to different interpretations of the specifications, lawsuits with their attendant cost, and the perpetual chance that, in spite of inspection, poor work will be done by the contractor in order to secure a profit on an underbid item. The contractor will be sure of a reasonable return on his investment in plant, he will not have to pay for special plant of use to him only on that one job, or to carry the burden of the gamble on unknown or unknowable factors.

The gamble belongs to the owner who gains or loses by the location of the site, the season of the year in which the work is done, the state of the market for men, materials, and money, and circumstances other than strictly construction work, which affect the cost of the structure; and when contractors are relieved of this gamble, they can bid closer, they can put their energy and knowledge into efforts to turn out good work, and not be constantly on the watch to cheapen the work for their own benefit—maybe salvation—and the owner's harm.

It will be for the owner and his engineer to say where to cheapen, when to spend, to pay extra prices, if they wish, to speed up work, or secure quick delivery. The job is the owner's; he should handle it and take the profits and the losses.

There can be, of course, great dishonesty under "cost plus" contracts, but, if this occurs, it is the owner's fault in lax supervision of his own forces or from incompetent or dishonest engineers.

It is hoped by this paper to obtain an expression of opinion as to the wisdom of substituting "cost plus" contracts for the other forms. At the present time, municipal work is usually required by law to be let to the lowest bidder, and any violation of the custom, even when the law permits, affords a basis for accusations of graft on the part of the officials responsible, no matter how self-evident the lack of ability of the low bidder may be.

If "cost plus" contracts can wisely be substituted for the other forms, the writer conceives it to be a timely occasion to endeavor to have the laws amended so that municipalities as well as individuals can

take advantage of it, and also to draft a standard form of contract sufficiently elastic to be adaptable to all usual work, so that the possible disadvantage may be reduced to a minimum and that dishonest officials may be prevented from discrediting a method which, in the hands of honest men, can certainly produce economical, rapid, and good construction.

DISCUSSION

Mr. Lavis.

F. Lavis,* M. Am. Soc. C. E. (by letter).—The author is evidently much impressed by the virtues of the so-called "cost plus" forms of contracts and the defects of the "lump-sum" and "unit" forms, but also evidently has a suspicion that there is a "nigger in the woodpile somewhere". It would seem to be almost self-evident that all three forms of contracts have their use at appropriate times and places.

The fact that the "cost plus" form has been used so frequently during the last three or four years, shows its adaptability to abnormal conditions, such as those which have existed during the war period when everything has been so unsettled that no one could conservatively venture a prediction as to the future. During this period, not only the Government, but manufacturers and others have required works to be carried out almost regardless of cost. It was necessary to employ existing organizations of both contractors and engineers and simply foot the bills, whatever they might be, and it is a high tribute to the honesty, integrity, and ability of both to know—as we do—that, on the whole, practically no undue advantage has been taken of this situation by either class and that most of the work has been carried out as efficiently and cheaply as was possible under the circumstances.

This state of affairs, however, is abnormal, and the methods used to meet it are seldom applicable to normal conditions. Usually, every one, from the builder of the smallest house to the greatest capitalist and investor, wants to know pretty closely what the cost is going to be before they start, and they are almost always willing to let the contractor take the gamble, if there is any. Often, however, he has had sufficient experience to be fairly sure in cases which would be pure gambling to others with less experience.

The author rather emphasizes the fact that, if the contractor has to take chances, he endeavors to cover himself by making an unduly high bid, but, on the competitive basis, this is at least partly controlled by reason of the competition. It is to be noted, however, that modern contracts and specifications for work to be carried out on a unit price basis reduce the chances the contractor has to take to a minimum. Splendid examples of this are the contracts used for the construction of the Catskill Aqueduct, by the Board of Water Supply of the City of New York.

The author points out the difficulties in regard to definitions, etc.; in other words, lack of skill on the part of engineers to describe what they want. Of course, it is true that difficulties do occur in the interpretation of contracts and specifications, no matter how skilfully they may be drawn, but many of these could often be avoided by the exercise of a certain amount of common sense on the part of the interpreters,

* New York City.

and, in any event, it is doubtful whether the causes for disagreement are more, or more often evident, in contracts based on unit prices than on the "cost plus" form, provided both are properly drawn and honestly and conscientiously carried out. Mr. Lavis.

Some of the objectionable clauses which cause trouble in connection with unit price contracts, pointed out recently by J. W. Rollins, M. Am. Soc. C. E., are as follows:

(1) Disclaiming responsibility for soundings and other information as to the character of the work.
(2) The insertion of the clause which may make any contractor's estimate and bid worthless, that is, "or as otherwise directed."
(3) Making contractors responsible for work for which the engineers make the plans, or requiring the approval of the engineer to any made by the contractor for temporary work, and in which the engineers reserve the right to control entirely the manner and means of construction.
(4) Holding the contractor responsible for work which has been built under the direction of the engineer, and has passed inspection, unless fraud and gross negligence is proved.
(5) Making the engineer the sole referee in settling all claims.
(6) The right to stop work, or any part of it, if it is to the interest of the engineer so to do, without allowing the contractor any compensation for such action.

The author points to similar clauses as increasing the cost of work carried out on a unit price basis. They do cause such increases, but they are indications of lack of engineering skill and ability, and should have no place in a properly and fairly drawn contract.

In considering the merits of any form of contract, it must be assumed that the same conditions will apply. For instance, it is useless to say that because the owner refuses to make proper sub-surface investigations, the unit price bid is a gamble. Under these conditions, any form of contract is a gamble.

The author thinks the contractor who bids in competition may bid high because of improper estimates of quantities. This is a confession of lack of sufficient engineering, or lack of engineering skill, the results of which can hardly be charged against a form of contract. If similar lack of skill is shown in connection with a "cost plus" contract, the result would be equally disastrous or expensive to the owner, if not more so.

Judging from some paragraphs of the author's paper, one would imagine that all bidders on unit price work were gamblers with the dice loaded in their favor, but the writer is sure that an extended acquaintance with the financial results of their work, year in and year out, would show that they make less net profit perhaps than the

Mr. Lavis.

majority of men in business who bring to their work an equal amount of skill, intelligence, and really hard work.

One thing must not be lost sight of in connection with "cost plus" contracts, that is, no matter how honest and conscientious the principals are, it is difficult to determine whether the actual output per unit of the men's time is as great on work done on this basis as it is on the lump-sum or unit price basis.

The theory that the engineer in charge of a contract is a referee or arbitrator between the owner and the contractor, the writer thinks will hardly hold in the Courts. As a matter of fact, engineers are so by courtesy, because usually they are held in high esteem by both their employers and contractors, and both, in many cases, are willing to abide by their judgment.

The author believes that with efficient honest contractors and engineers, better results can be obtained under the "cost plus" form of contract because "the owner will not be confronted by bills for extras, vexatious claims for adjustments due to different misinterpretation of the specifications", etc.

With honest capable contractors, there is seldom much trouble. The author must bear in mind, however, that with the unit price contract all the engineer has to do is to see that the quality of the work is good and that its performance is up to date. With the "cost plus" form, notwithstanding the reputation of the contractor—and with many it is very high—the owner's representative, if he is conscientious, will not only satisfy himself as to the quality of the work, but also of the honesty and efficiency of the contractor's organization. He can no more take for granted that pay-rolls are not padded, that outside commissions are not paid, or that the men or the organization are efficient, than he can take for granted the depth of a foundation, the quality of the soil, or the acceptance of Portland cement or steel without testing.

The author asks the specific question as to the desirability of adopting the "cost plus" form for municipal and other public works. Under war or similar conditions it may be; but, for normal conditions, no. The writer has had some experience and vexation in connection with the red tape, etc., thrown around the letting of municipal or other public works contracts. The award to the lowest bidder is often a cause of expense, but, by and large, and taking into consideration all the facts, it will be found that there are many good reasons for the various regulations and safeguards thrown around the actions of municipal and governmental officers in the expenditure of public funds.

Besides all these things, there is also the important item of predetermination of total cost. Of course this is accomplished to a certain extent in those forms of the "cost plus" contract which name an

estimated total cost with a sliding scale fee increased for keeping within the total, and decreased for exceeding it, but there is not the same assurance—which is often desirable—with this form as with others. Mr. Lavis.

It seems hardly feasible to define exactly what kinds of work or what set of conditions warrant the adoption of one or another form of contract, but, in general, the following may be suggested:

a.—For certain forms of work, involving clearly defined materials and easily determined quantities, where important changes are not probable, and where the size of the job is not too large, the lump-sum form of contract is indicated.

b.—For forms of work, where specifications can be drawn, which will define reasonably clearly each class of work to be performed, but where only approximate determinations of quantities can be made, the unit price form is indicated.

c.—When conditions are such that the work to be done cannot be defined reasonably clearly and definitely, or when the period of construction extends over such a long time that it is difficult to forecast conditions so far ahead, or in a time like the present, when everything is so disturbed, there is little option but to adopt the "cost plus" form.

The writer realizes, of course, that these conditions overlap, and that many others may influence the decision, but, in this, as in everything else, the skilful experienced engineer will realize the conditions, and, being informed as to all the different kinds or forms of contracts, will advise his client accordingly.

Missac Thompson,* Jun. Am. Soc. C. E.† (by letter).—For purposes of this discussion, the writer will classify contracts as long-time and short-time. Without trying to draw a line of demarcation, he will state that the long-time contract is one, the fulfillment of which will take sufficient time to anticipate marked changes in the market prices for labor and materials; whereas, the short-time contract is one, the fulfillment of which will not take sufficient time to make appreciable difference in the market prices of commodities for the particular work. Mr. Thompson.

This division cannot be made in terms of time measure alone, other circumstances, such as the political and economical situation of the country, the particular contractor, and his organization, are also factors to be considered.

Long-time contracts, if based on preliminary designs, are not often drawn on the lump-sum basis. If such contracts are drawn on the basis of item charges, again, both sides are gambling.

It is very difficult, if not impossible, to define what might be acceptable material or workmanship. In such construction as rein-

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† Now Assoc. M. Am. Soc. C. E.

Mr. Thompson.

forced concrete, with the ordinary method of inspection, the owner or his representative has no practical protection against dishonesty or negligence on the part of the contractor.

It seems to the writer that, in long-time contracts, the interests of both parties—owner and contractor—will be safeguarded, if all the materials forming a permanent part of the job are furnished by the owner, and the contractor simply bids on the actual construction on a labor-day basis for the unit of work.

This mode of contract can be illustrated by an example: Suppose Jones wants to build a reinforced concrete factory, and Smith is the contractor for the job. Jones furnishes all the permanent materials for the work; Smith bids on furnishing the timber for forms and erecting the form work at one-fifth carpenter-day per square foot of form; or, on mixing and placing the concrete at one-half labor-day per cubic yard, etc.

In some cases, the contractor by superior knowledge of market conditions may obtain materials to better advantage. In that event, his services may be made use of and paid for as such.

The owner by furnishing the materials is sure of getting what he wants. He is relieved of rigid inspection obligation, because faulty workmanship will always show. He can get more competition for his job, because much less outlay of capital on the part of the contractor is necessary. In most cases, the owner, with his ready cash, is in a better position to bargain for the materials.

On the other hand, the contractor is relieved of probable rejections, disputes, etc.; he saves time by doing away with a lot of unnecessary checkings, markings, etc.; and he is safeguarded against wage increases, strikes, etc.

In some cases, the same basis may be used for short-time contracts as that suggested for long-time contracts. However, in many cases, this procedure is impractical:

First.—The owner usually finances the work by borrowing from banks or individuals, and the loan cannot be arranged to pay for materials as they are delivered.

Second.—It is more practical for the owner to pay somewhat more and know beforehand the cost of the work, in order to arrange for his financing.

Third.—Very close estimates of materials and labor can be made beforehand, and the contractor's percentage of profit on small jobs is necessarily larger and the elements of uncertainty are less.

Fourth.—Even if bids are taken from preliminary designs and estimates, a clause may be inserted in the contract terms, by which additions and subtractions may be taken care of.

It seems to the writer, therefore, that, for short-time contracts, the lump-sum basis is practical and desirable. However, the "cost plus"

definite fee or the cost plus percentage method of contract is objectionable, because it kills competition which is the key to efficiency. It offers undeniable opportunity for favoritism on the part of the owner's representative. Moreover, it is out of the question to persuade most investors, who borrow to finance their enterprise, to let a contract on the cost plus percentage basis. Mr. Thompson.

F. H. Frankland,* M. Am. Soc. C. E.—The author states that he does not pretend to present arguments either for or against any particular form of contract, but a reading of the paper shows that it is only a brief for the "cost plus" system. In fact, in the author's remarks just preceding the opening of the oral discussion, he admitted that this was true. Mr. Frankland.

Most of the worst features of lump-sum and unit-price contracts are discussed in the paper, whereas nothing objectionable is mentioned as relating to the "cost plus" system. In the speaker's opinion, based on a great many years' experience in the supervision of engineering construction, all the ordinary forms of contract, as exemplified in lump-sum, "cost plus", and unit-price work, have serious drawbacks.

In one place, in his paper, the author makes the serious charge that the engineer is always biased, and no matter how hard he tries to be fair, no matter how cordial the relation between himself and the contractor may be, in case of argument the latter bears the burden of proof. This is a pretty strong statement, and the speaker considers it a somewhat unwarranted aspersion on a large number of engineers, possessed of experience, principle, integrity, and (to use the Shakespearian expression) engineers with "guts", who are absolutely without bias against contractors. Many instances could be cited where supervising engineers have fought hard and successfully for a square deal to a contractor.

The speaker mentions this because it is one of the stock arguments which is always brought up, when "cost plus" contracts are discussed. The war is over, and conditions have changed more suddenly and more completely in a given time than ever before in the recollection of most of the members of the Society, and the present seems to be the psychological moment for the engineers and contractors of the United States to "get busy" and do something about forms and bases of contracts. It is generally admitted that a large amount of foreign engineering and contracting is now open to American engineers and contractors, if they will only go after it in the proper way.

One of the serious disadvantages of the ordinary kind of "cost plus" contract is the lack of incentive for the contractor to complete the work at the lowest legitimate cost. The ideal form of contract should provide for the best possible workmanship at the lowest legitimate cost.

* New York City.

Mr. Frankland.

The ordinary forms of "cost plus", lump-sum, and unit-price contracts do not fulfill these requirements, and the time has arrived when something should be done toward improving the methods of letting construction contracts now in vogue. It must not be imagined by any one that an ordinary "cost plus" basis for foreign contracts will be acceptable to the owners. This form of contract is rightly looked on abroad with suspicion.

If the whole truth was known regarding some of the "cost plus" contracts let in the United States during the past few years, this form of contract would "smell to high heaven." Of course, we all know that it was absolutely necessary to let a great deal of work during the war on a "cost plus" basis in order to get it done, but to the speaker that does not seem to be a valid excuse for the continuation of that form of contract.

Mr. Smith.

J. Waldo Smith,* M. Am. Soc. C. E.—The speaker is one of those who believe that, under certain conditions, there is a proper use, and even necessity, for each of the forms of contracts outlined by the author, although he does not at all share the opinion as to the desirability of the general use of a "cost plus a percentage" or "cost plus a fixed sum" contract, or the difficulty of carrying out a regular itemized form of contract without including "extra work."

If the plans are so indeterminate and the investigations so incomplete that it is impracticable to form an estimate of the total cost within reasonable limits, then, without doubt, the "cost plus a percentage" contract is the only one which fits the case, especially if great expedition is required. If, however, it is possible to make an estimate of the total cost within reasonable limits, then some form of the "cost plus a fixed sum" contract is entirely practicable and to be preferred to the "cost plus a percentage" contract.

If work was to be done generally by either of these "cost plus" forms of contract, it would certainly lead to the grossest discrimination and favoritism, would prove an insuperable obstacle to the proper development of the contracting business, and tend to destroy the incentive for the study of new methods and the exercise of the greatest resourcefulness in conducting the work. It would also exclude the gambling element which has so strong an attraction for the average contractor.

Under the conditions which have prevailed during the past two years, and the enormous volume of work done without the preparation of detailed plans, where many of the conditions were indeterminate and where great expedition was required, it was undoubtedly necessary to use both the "cost plus a percentage" and some form of "cost plus a fixed sum" contract. In the majority of cases, it is believed that the general type of contract used by the Construction Division of the

* New York City.

U. S. Army, of "cost plus a sliding percentage with a limiting sum", or one of its variations on the same principle, was entirely practicable, but there was little need for a "cost plus a fixed percentage" contract. Mr. Smith.

The author, both in his paper and in his remarks when presenting it before the Society, seemed to have a decided leaning toward a "cost plus" contract for ordinary use. For engineers to advocate such a form for general use is, in the speaker's opinion, a confession of weakness—that engineers are unable to draw contracts and specifications, founded on ample investigations and full information, which are fair to both parties to the contract and on which bidders can make an intelligent and safe proposal.

It is well known that engineers in charge of public work are handicapped in drawing contracts as fair to the parties thereto as they desire, by reason of limitations imposed by the law departments of municipalities or States. These restrictions are largely the result of previous experiences in Court, and the legal authorities seek, by injecting arbitrary clauses into the contract, to prevent recoveries in the future, rather than to make the contract so fair between the parties to it, that there would be in general no incentive to bring action thereafter.

The engineer is often confronted, in the preparation of plans, with a great cry from principals, who may be either private individuals or public officials, to see the actual construction work started before the necessary investigations have been completed. Nothing is more vicious or sure to lead to trouble afterward.

If the contractor is not given sufficient information, if the known difficulties are not set forth, if the proper investigations have not been made by those who have been on the job for months, if it is not clearly stated just what is to be done under each item and how payment is to be made, and if everything reasonable is not done so that the contractor can size up accurately the conditions with which he will probably be confronted, and submit an intelligent bid, trouble is almost certain to develop during the progress of the work and cause litigation afterward. If, however, all this information has been obtained and spread before him, and he is given a fair show and not treated arbitrarily, not only would he fail to collect much through the Court in case of litigation, but the average contractor would not even bring legal action.

Edward F. Hammel,* Jun. Am. Soc. C. E.†—The technical press has been devoting a great deal of time and space to this subject of contracts, and all engineers have had various experiences with the matter of contract forms. The speaker will limit his remarks on the subject very largely to building work, to which his experience has been confined. Mr. Hammel.

The speaker asked four contractors to bid on a certain job, and of

* New York City.
† Now Assoc. M. Am. Soc. C. E.

Mr. Hammel.

these, three wanted the work on a "cost plus" basis. Since the owner wanted to know how much the job was going to cost, this form of contract could not be considered. This owner knows how much he can reasonably get for the floor space of the building which he proposes to erect; he also knows about what his taxes and operating costs will be, and, in the light of these facts, he knows about how much he can spend on the erection of the building and still obtain a fair return on the investment. If he cannot get enough interest on the investment he proposes to make, he will not feel warranted in going ahead with the operation. Finally, one of the contractors returned the plans and specifications, stating that, due to the present unsettled conditions of labor, he was unable to undertake the work on any other basis. After arguing on the merits of the "cost plus" contract, the other two finally agreed to furnish lump-sum bids, but stated that ultimately it would cost the owner more than on the "cost plus" basis, because they would have to add a large figure for contingencies, which might be avoided if they went ahead on a cost plus a fixed fee or a cost plus a percentage basis.

It seems to the speaker that the various forms of contracts which have been drawn, have been designed to fit special conditions, and as long as the labor market was fairly stable and wage agreements were adhered to by labor—there have been agreements in the past, which have lasted a certain time—and as long as the prices of materials were stable, there seemed to be no particular reason why a contractor could not submit an estimate on the lump-sum basis and finish the job with a reasonable amount of profit. Certain contingencies might arise which would reduce that profit, and others which would increase it, but the contractor was reasonably sure of finishing the job with his overhead covered and a certain amount of profit.

During the war period, conditions were entirely different. The Government speeded up its work, because the buildings had to be erected almost immediately. If they had been erected a year later, they would not have contributed anything toward winning the war. The Government officials were not able to make exhaustive investigations; they did not have the data, nor the time to get them, and it was absolutely essential that some form of contract should be adopted, which was fair to the contractors, otherwise they would not have taken the work. The only way out of it, apparently, was to tell the contractors to build the necessary structures, etc., on some form of "cost plus" contract. The Government chose contractors who had organizations large enough to do the work on a speedy basis, and said: "We will pay you for the cost of the work and for your services", and it seems that the "cost plus" contract was the only form which could have been adopted to meet the emergency.

Addressing a man who was on the contractor's staff on one of the cantonment jobs, and who spent a good part of his time riding to and

from the city in an automobile, the speaker said, "You must be making a lot of money". To which he replied, "Oh, no, the Government is paying for this". That, perhaps, was a little item, financially, but it was an indication of the wastefulness practiced on "cost plus" jobs. At times, something has to be sacrificed, and the Government made the sacrifice of money in order to obtain speed. Mr. Hammel.

It may sometimes happen that the owner of a manufacturing plant, such as an automobile factory, may want to extend that plant. He is very anxious to get his addition completed, and he does not care if it costs 10% more than the possible lowest cost, provided he can get it done speedily; therefore, it may seem to him that the "cost plus" is a fairer contract, and, perhaps, it is under these special circumstances. He is willing to sacrifice a financial saving in order to have the building put up more speedily.

On the other hand, a man who is contemplating building on a certain lot, and investing a certain amount of money as an economical investment, can afford to wait a little longer and have his building put up in accordance with a complete set of plans and specifications.

It seems to the speaker that the advocates of the "cost plus" contract miss certain points. They state that for the contractor it is the only fair method; that building contracting is a gamble; that it is so much of a gamble that the owner ought not to expect the contractor to make a fixed bid; and that he (the owner) ought to take a chance on a "cost plus" contract.

There is no more of a gamble in contracting within certain limitations, than in any other field of business. If the men engaged in the building business were generally to adopt the "cost plus" form of contract, an era of uncertainty would be reached in the cost of buildings, which would greatly restrict construction operations.

The gamble, as the speaker sees it—possibly he may be wrong—is almost entirely related to labor costs. There is no reason why the architect or engineer cannot prepare a complete set of plans and specifications and cover every item. There is no reason why an engineer or quantity surveyor cannot study these plans and specifications and take off accurately the quantities of materials which will be necessary to complete the structure, as accurately, or more so, than the engineer can compute his loads and stresses; and there is no reason why the contractor cannot take this as a basis on which to compute his construction costs. Then all the contractors will be bidding on the same quantities, and not on quantities which may vary greatly, due to erroneous estimations by their own staffs. They will all use the quantities prepared in the engineer's office, and the only thing that will vary is the cost of the labor necessary to put those quantities in place.

It seems to the speaker that some basis of adjustment, due to changes in labor wage scales, can be made. Let the contractor figure

Mr. Hammel.

on the current labor prices at the time he is making his estimate. He can order his building materials and cover himself on this cost for some time in advance. Future contingencies as to labor costs might be met by an additional stipulation in the contract; for instance, if a provision was put in the contract that if the cost of labor is increased, or if the present or prevailing rates are increased by circumstances not within the control of the contractor, the owner agrees to stand all, or perhaps a part, of such increase, so that the contractor would not lose much on that score, and the owner would be paying only for what he gets.

Then, there is the matter of incentive. As a matter of fact, the speaker knows of some jobs where the lump-sum is favored at the expense of the "cost plus" job. If a contractor is handling three or four jobs, and one happens to be on a "cost plus" or percentage basis, and the others happen to be on the lump-sum basis, and he has any bricklayers who are able to lay more bricks than others, he will keep his good men on the lump-sum jobs and send the "cripples to the hospital"; that is what these "cost-plus" jobs have been called. Three or four men get on the end of a rope, and somebody asks, "What are you doing there?" and finds out that all they are doing is holding the end of the rope; that is the way it goes on the "cost plus" job; there is no incentive to reduce costs. On the lump-sum job, however, the incentive to the builder is to get his work done at as low a cost as possible; he is not going to let his men loaf.

To be consistent the "cost plus" contractor should let his subcontracting work on the "cost plus" basis, but he does not do that. On the average every building contract has a general contractor and, perhaps, 15 or 20 sub-contractors, say, there are 15. The general contractor will often handle his own masonry and carpenter work. The speaker asked one of these contractors who was arguing that it was not fair to ask him to bid on a lump-sum basis, "How do you do with your sub-contractors? How about these fellows? How much work do you do yourself?" He replied, "We do 30% of this ourselves." Frequently, however, they do not do more than 20% of it. The speaker then said to this contractor, "Of course, you work on a percentage basis with all your 'subs'?" He replied, "Oh, no." The contractor will not show that element of fairness to the "subs" which he asks the owner to show to him. He would be willing to tie every one of the sub-contractors down to a lump-sum contract, and whether or not they were losing money, he would hold them to the letter of these contracts. If they are losing money, he says, "If you won't finish up under this figure, the surety company has got to make good on it", while he himself is safeguarded against any possible loss. If the "cost plus" contract is to be adopted, it ought to benefit more than the general contractor who is often a

looker-on in this game; it ought to be parceled out to every sub-contractor on the job. Mr. Hammel.

There is one other thing: If we carry this form of contract to its ultimate conclusion, who is going to get all these percentage jobs? There are certain contractors who have no organizations, and they depend on those who have to build the work, to get it out at a lower cost than their competitors; and they often get jobs on that basis without having the organization to complete them. If this lump-sum type of contract that is so unfair, is abolished and the "cost plus" or percentage contract is universally established, who is going to get these nice "soft" jobs? The "inside" man is the one who will be favored. An owner is not going to ask half a dozen contractors to bid as on a "cost plus" job. He is going to pick one man and engage him to handle the matter. The result will be that all the contractors who could go in on a competitive basis to get the business, simply will not know of the work or the owner. Gradually, they will disappear, and after awhile only a few of these favored contractors will be left, who will be making all the money. If a competent young engineer with a little money, some practical experience, and a lot of ambition, wants to get into the game, there will not be any use of his attempting to do so, because he will be "squeezed" out. He may be a good competent man and have a lot of push, but he will not get anywhere. He cannot go out on the basis of merit and get a job; he will have to "know" somebody in order to be favored. His early years must be spent in making acquaintances.

It seems to the speaker that, after all, under the "cost plus" contract, the contractor ceases to function as a business man. He becomes, not a man who has to use his shrewdness, his executive ability, and his brains to maintain an efficient organization and keep its operation within a reasonable cost, but simply a supervisor of labor, a superintendent of construction, and a material purchaser, and, as such, is very highly paid. All he has to do is to see that the other fellows do their work. He parcels out the work to others and looks it over. He will have degenerated from his present position. Sooner or later, he will be found out, and then he will be found out of a job, for there is no reason why the engineer cannot supplant him. He can sublet what work he wants to, purchase the materials, and supervise the construction for his client; he can add that much to his fee, and then, perhaps, engineers will be able to make a reasonable living.

Lazarus White,* Assoc. M. Am. Soc. C. E.—The speaker believes that engineers have an idea that all contractors are in favor of the "cost plus" form, because it makes an easy and safe job for them. He believes that this paper, as he reads it, advocates the "cost plus" system. Mr. White.

* New York City.

Mr. White. at least, it gives arguments in favor of it and arguments against other forms of contracts.

The speaker has had experience both as an engineer and a contractor, and this experience has led him to believe that the old form of letting contracts on a competitive basis is much the best. The great objection to the "cost plus" contract is the means of getting it. There is a moral question which outweighs the question of cost. If the public in the long run is getting something that is not right, they are the ones who suffer. There is something straightforward in letting a contract on a competitive basis, if the job is properly prepared, say, like the contracts of the Board of Water Supply of the City of New York, and the contractors are invited to bid.

They form their own opinions as to what it is going to cost them; they put up a bond; the man who is low is taking some chance, and if the owners have not a very good reason for turning him down, he ought to get the job. Suppose the Board of Water Supply, through its chief, wanted to get a man on a "cost plus" basis. How would he do it? The speaker believes he would not know how to do it. The usual way would be to have a lot of people running around telling what great men they were, that they had the organization, etc.

When a man gets a job, he intends to make the best showing possible. What kind of work will he get from subordinates? How long does it take them to realize that they are on a "cost plus" basis? Every superintendent is met by this question, "It is not coming out of your pocket, is it?" And the same argument comes from laboring men.

A contractor on a "cost plus" job is not a contractor, he is an accountant; and nine-tenths of his time is given to demonstrating that he paid right prices for everything that he bought, and accounting for every cent expended; and that is the principal thing. It is not a question of efficiency any more; that is swallowed up. There is ten times more accounting on a $1 000 000 contract at "cost plus" than there would be on the same $1 000 000 of straight contracting.

Besides it eliminates the gamble. If a man takes a $1 000 000 contract (it is attractive, it must be said), he may make $100 000. In an ordinary contract, with good management, but on a "cost plus" basis, he is a few thousand dollars ahead of the job at the end of a year's hard work trying to demonstrate that he is not a crook.

There is another inherent difficulty: If the "cost plus" was carried to its logical conclusion, nobody would know what anything would cost, because certainly the total would be the sum of a number of "cost plus" jobs; and what would that total be? It seems that, on public work, in the long run, the "cost plus" contract could not be tolerated, because public work is done against definite appropriation on a "cost plus" job. When a city lets a job for $1 000 000, it is committed to

$1 000 000. When it lets a job on a "cost plus" no one knows to what it is committed. The same thing applies to the individual. Mr. White.

The speaker is simply giving his experience as an engineer and a contractor with both styles of construction.

ARTHUR P. DAVIS,* M. AM. SOC. C. E.†—The form of contract suggested by J. A. L. Waddell, M. Am. Soc. C. E.,‡ is excellent, if it can be carried out, but its use requires that all details be thoroughly well known in advance. Mr. Davis.

Now, those who have had experience on very large jobs, realize that it is very difficult in all such jobs to know the details in advance, and it is absolutely impossible in others. For instance, how is any contractor going to know all the details of a job on a foundation that has to be put down 100 or 150 ft. below the level of the ground, until it is actually opened and dug out. He can do all the boring, and all the guessing, he pleases, and yet he does not know how it is going until it is open. Such work always involves a big element of risk, as does also every long tunnel that passes through a high mountain. However closely it may be examined, there is a large element of uncertainty in the conditions that are to be encountered by a contractor who takes such work on any kind of a basis except a "cost plus" basis.

The term, "cost plus", is very indefinite. There are two radically different methods of "cost plus" contracts. One is to pay the cost plus a percentage and the other is to pay the cost plus some fixed sum. They are radically different, as different as the east is from the west. The speaker recognizes the fact that there were conditions at the breaking out of the war, which had to be met by rather unusual methods; but he has talked with both engineers and contractors who were engaged in the "cost plus" work, and has yet to find one who could convince him that it was not possible to have let those contracts on some other basis than to place a premium on extravagance and waste. In other words, to find out closely enough so that some fixed sum could be named as compensation for the use of the contractor's skill, equipment, and organization; or, still better, that some method could not have been outlined by which he could have received just compensation, and, in addition, a reward for properly ascertained economies—economies both of time and of money.

If a contract is let on the basis of cost plus a fixed sum, the contractor will earn his fixed sum sooner if he rushes his contract through and closes it up in a short time, which is the great consideration in the case of preparation for war. Deliberately, that incentive was taken away from the contractor.

We regard the great judges of our all-powerful Courts among our highest class public officials. They should be, and usually are, selected,

* Washington, D. C.

†Now President, Am. Soc. C. E.

‡ *Contracting*, September 15th, 1919, p. 178.

Mr. Davis. with great care; men who are just, who have an incentive higher than personal gain, as far as human beings can be found who do; and yet we find it to be a universal rule in all the Courts, from the highest to the lowest, that a judge shall not be permitted to sit in judgment in a case in which he is interested, because it is humanly impossible for him to eliminate his personal interest from his actions and his determinations.

Now, that is just as true of contractors as it is of judges. Even the contractor's own subordinates, his own employees, know that he has interests a certain way; that it is to his interest to make it cost more, because in that case he gets a larger fee; and temptations are such that it is absolutely necessary to guard against contracts of that kind. Assuming, however, that those who know more about the circumstances than the speaker, are right, that the "cost plus" contract was unavoidable in the haste of preparations for war, let us not lend any influence to extend that pernicious practice in ordinary work. Let us find out what the work should cost, as nearly as possible, and where it can be determined well in advance, the unit price contract, with fairly drawn specifications, is, as far as the speaker's experience goes, the fairest of all methods.

In that case, engineers have the opportunity of reasonable changes without affecting materially the profits of the contractor, if he has made a well-balanced bid; and in the case of earthwork, at least, and many other kinds of work, it is impossible to tell in advance how each of the units is going to run. A classification will be carried on as the work progresses, and the different classes put on a unit basis, so that the errors of estimate in advance will not greatly affect the profits, because where the material is harder, it commands a higher price for a higher classification.

It is at least logical and practicable, and is likely to make it to the interest of every man connected with a piece of work, to do the work right, and do it economically; and let us not assume that people are superhuman, and do not, and will not, look to their interests in getting out their work.

Where work is of such uncertain outcome, or is likely to involve many changes or much hazard, a contract let on a cost basis with compensation to the contractor, to be paid after a manner similar to that suggested by Mr. Waddell, is the best solution, even if the uncertainties leave more uncertainty as to the contractor's compensation, which is not nearly as bad as to leave him with a possibility of heavy loss, or to offer him a substantial premium for extravagance and waste.

Mr. Quimby. HENRY H. QUIMBY,* M. AM. SOC. C. E.—A large proportion of construction contract work is done for the public—the owner is the public, either Municipal, State, or National—and, in such work, competition

* Philadelphia, Pa.

is essential. Is there a practical and satisfactory scheme for securing competing bids on a "cost plus" basis? A limit of expenditure in every such contract is also essential, and as the limit must be fixed before the contract is delivered, and, therefore, before the work is done, an under-estimate might cause the serious embarrassment of reaching the limit before the completion of the necessary work. Mr. Quimby.

"Cost plus", whether it is plus a percentage or plus a lump sum fixed or variable, sounds fair, and if by it the owner gets just what he pays for, it is fair; but we must conclude from what we see and hear that that result is not commonly secured. If the contractor's compensation increases directly with the cost of the work, the system puts a premium on extravagance, as Mr. Davis has stated. It certainly may operate to eliminate all ambition on the part of the contractor to show his superior skill and efficiency as such, even if it does not induce him to nurse the job.

Only a few days ago, a division engineer complained to the speaker of the slow progress being made on the force account portion of a certain contract, and declared that he always noted a marked difference between the spirit in which the percentage portion of the contract was done and that in which the work was performed on the fixed-price portion by the same men. When it was suggested that the men need not know that the work is on a separate account, he replied that the men always know, and invariably produce less.

The sliding scale of compensation based on the relation of the actual total cost to a guaranteed maximum sounds logical, as tending to induce effort and promote efficiency, but a guaranteed maximum cost to the owner necessarily involves a risk of loss to the contractor, to avoid which is one of the arguments for the "cost plus" idea. Also, in case competition for the contract is necessary—as for public work—the sliding scale for compensation would have to be prescribed in the specifications, and the bidding would have to be on the guaranteed maximum, the temptation to take chances by a bidder being, therefore, much the same as in straight lump-sum contracting.

As the guaranteeing of a maximum cost will require the bidder to make a careful estimate, the plans must be definite as to the extent of the work and as complete as for a straight lump-sum bid, and this requirement eliminates one of the principal arguments for the "cost plus" contract.

If the bidder must take a risk by guaranteeing a maximum sum, why should he not have all he can save out of it; and if he must share such savings with the owner, why should not the owner share any loss that may result through an excess of actual cost?

In "cost plus" work, the necessity for checking the contractor's costs involves not only the additional cost of services, which may be quite appreciable, but is likely to occasion disputes and develop unpleasant

Mr. Quimby. inquisitorial procedure, leading to direct supervision by the owner's representative of all the operations under the contract—purchase of material and direction of work—a situation under which a real contractor will not be happy, and which, pursued logically, will tend to dispense with his services. In the Graff system, described in "Bridge Engineering", the contractor stipulates that he "be left absolutely untrammeled and allowed to enter the buying market and otherwise to conduct his operations as though he were executing a straight contract."

There is a form of contract which, although giving the contractor an opportunity to develop his best methods of operation and win both the contract and a profit on it by his skill and energy, provides reasonably for both changes in plans and some kinds of contingencies. This is a lump-sum contract for the definite construction shown by the plans, together with unit prices for the various classes of work and material involved, these prices being applicable to changes in quantities—in full to additions and a reduced amount, say, one-half, to deductions. For any class of work not covered by such unit prices, a force-account item is also provided, the percentage to be bid in the proposal the same as the other prices. An aggregate of a bid for comparison of bids is obtained by extending the unit prices and force-account percentage on assumed quantities, and adding the extensions to the lump-sum bid for the work shown; thereby competition is secured throughout. In the event of changes being desired, which the unit prices do not specifically cover, or which create conditions not contemplated and make the bid price unfair, or, in case obstacles are encountered for which the contractor has not assumed responsibility, a special dicker is necessary, and, in such a dicker, the party of the second part to the contract is ordinarily quite as potent as the party of the first part.

The plans ought to be as complete as possible before the contract is made, and if an owner is not so prepared and goes ahead on meager data, he must expect to pay more in one way or another. If the plans can be made definite, so that quantities can be determined and under-surface conditions be reasonably explored, the greatest satisfaction to the owner, engineer, and contractor will generally be secured by a lump-sum contract or a fixed unit price contract with equitable provision for adjustments.

Mr. Perry. J. P. H. Perry,* M. Am. Soc. C. E.—The impression gained by the speaker from the discussion on this subject, seems to be a fairly common one, namely, that the cost of work, that is, the unit cost under the "cost plus" form of contract, is higher than for similar work done under a lump-sum or straight contract basis.

The Turner Construction Company, of New York City, of which the speaker is a Vice-President, has had an experience which would

* New York City.

contradict that impression. The Company is in the building business and has erected nearly 600 reinforced concrete factory, warehouse, and other industrial buildings since May, 1902. On this work it has been found consistently that lower unit costs have been obtained on jobs done on a "cost plus" basis than on those for which the contract was a lump-sum or straight contract. Of nearly 750 contracts which have been handled to date, the division between the lump-sum and "cost plus" types of agreement has been nearly "fifty-fifty." Mr. Perry.

At the request of the editors of *Engineering News-Record,* in the spring of 1919, the Company investigated its cost records to see what conclusions could be drawn from cold facts and figures as to the effect of the type of contract, that is, lump-sum or "cost plus," on unit costs of building work. In order to get the comparison, the Company had to go back to 1917.

During this year, it built 22 factories or warehouses of reinforced concrete, on each of which the contract exceeded $100 000. The average for each of the 22 jobs was $413 000, which makes each building a fair-sized building contract. Of these 22 buildings, 13 (averaging $470 000 each) were built under a "cost plus" form of contract and 9 (averaging $329 000 each) were built under a lump-sum contract.

By a remarkable series of coincidences, the buildings that were "cost plus" or lump-sum parallel one another in the outstanding features, which, on a critical examination of the cost records, might be thought to affect the costs. That is, for every "cost plus" job, the Company had another, generally similar in size and location, under a lump-sum contract. Similarly, for every beam and girder "cost plus" job, there was a beam and girder job on lump-sum, and, in the same way, flat slab jobs, brick veneer jobs, and in-town and out-of-town jobs matched up fairly well.

It has been the Company's experience that, for purposes of comparison of the cost of buildings, three labor units and a general expense item taken together serve as a very accurate criterion as to the relative cost of the complete building. These are, as itemized on the Company's accounting system:

Item 3-F.—This item includes the cost of carpenter and labor work per square foot of floor required for placing and removing floor forms, including all hoisting and miscellaneous work in connection therewith. This item does not include the cost of making the forms before they are used for the first time and, therefore, a comparison is possible as between buildings having a different number of stories.

Item 22.—The cost of labor per cubic yard for all work in connection with the mixing and placing of concrete is included in this item. An average of this unit will give a very fair comparison.

Item 30.—The cost of all labor per ton for receiving, handling, bending, and placing steel reinforcement is kept under this account.

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As the labor conditions under which this work is done, differ in the metropolitan districts from conditions existing outside of these districts, one comparison is made between jobs in the metropolitan districts and another between jobs outside of such districts.

Item 19.—This includes the general expense expressed as a percentage of the total pay-roll for the job. This item is included to show that the units obtained on Items 3-F, 22, and 30, were not influenced by incorrect charging of labor to general Item 19, thereby reducing the amounts properly chargeable to the cost of forms and the placing of concrete and steel.

The figures on these items for the jobs in question are shown in Table 1 which should be self-explanatory.

The speaker would like to call attention to certain factors back of the figures quoted in Table 1.

TABLE 1.—COMPARISON BASED ON A STUDY OF COST UNITS ON ALL FACTORIES AND WAREHOUSES COSTING MORE THAN $100 000, BUILT BY TURNER CONSTRUCTION COMPANY IN 1917.
Total Number, 22.

	Average unit on 13 percentage jobs.	Average unit on 9 lump-sum jobs.	Percentage jobs were cheaper by:	Lump-sum jobs were cheaper by:
Item 3-F: Cost per square foot of placing and removing floor forms	5.75c.	6.25c.	8%	.....
Item 22: Cost per cubic yard of mixing and placing concrete	$1.02	$1.04	2%	.....
Item 30: Cost per ton of receiving, bending, fabricating, and placing steel reinforcement on jobs in the metropolitan districts	(10 jobs) $15.71	(4 jobs) $14.89	.....	5%
Item 30: Cost per ton of receiving, bending, fabricating, and placing steel reinforcement on jobs outside of the metropolitan districts	(3 jobs) $9.01	(5 jobs) $8.87	.....	2%
Item 19: General expense. The amount charged to this miscellaneous item is expressed as a percentage of the total pay-roll.	1.8%	2.3%	.....	.....

Placing and Removing Floor Forms.—There is some difference in the cost per square foot for Item 3-F, as between flat slab and beam and girder construction. By a coincidence, it happens that 14% of the floor area in the percentage jobs was beam and girder and 86% was flat slab, while, in the lump-sum jobs, 13% was beam and girder and 87% was flat slab. Because of this similarity, the comparison made between the average units on the percentage work and on the lump-

sum work is not influenced by any difference due to the slightly greater cost of beam and girder forms over flat slab forms. The column spacing was sufficiently similar in all the buildings so that any difference due to this element may be disregarded. Item 3-F has a tendency to be lower in buildings having a greater number of stories, because the 3-F unit for the first floor is nearly always higher than the same units for the floors above that. As all the buildings studied are five or more stories in height, with the exception of two, which are four stories, the influence of the element of height is very slight. It is to be noted that the next to the lowest 3-F unit obtained on any of the 22 buildings was at Becker's Aniline and Chemical Building, which is only five stories high, and each of the units made on the two four-story jobs are below the average. It is believed that a comparison of the 3-F unit, as between the percentage group of jobs and the lump-sum group, is a fair one and means that, if anything, the organizations on the percentage "had the jump" on those on the lump-sum job for this item. Mr. Perry.

Mixing and Placing Concrete.—With equal efficiency on the two classes of construction, the averages of Item 22 should run very closely together, and such is the case, the difference being only 2%, in favor of the percentage organizations.

Receiving, Bending, Fabricating, and Placing Steel Reinforcement in Metropolitan Districts.—The lump-sum jobs exceeded the percentage jobs by 5%, and this difference is probably justified by the more difficult conditions affecting this Item 30, which existed on the percentage group of buildings.

Receiving, Bending, Fabricating, and Placing Steel Reinforcement on Jobs Outside of Metropolitan Districts.—The lump-sum jobs exceeded the percentage jobs by 2 per cent. The comparison is a fair one and should be a true indication of efficiency.

General Expense.—The amount of money covered by the classification in Item 19 is very small compared with the other items used in Table 1. The only reason for tabulating the item was to show by its smallness that improper charges had not been made to it, thereby reducing the charges of pay-roll made to Items 3-F, 22, and 30.

To some of the discussors, the results of this investigation made by the speaker's Company will sound like anarchy, but there is a sound reason back of these figures. It has been the experience of the Company that under a "cost plus" form of contract, there exists essentially a partnership between the owners and the contractor. There is only one interest, namely, to give the best work for the least money in the quickest time. If the contractor expects to continue in business, he has to earn the confidence and commendation of that particular owner. To lump-sum contractors there seems to be, in some cases, a

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different interest. Each side wants to be sure that the other side does not get the advantage under an agreed price.

Under "cost plus" contracts, the contractor feels free to go to the owner with suggestions for modified ways of doing the work, and for the substitution of methods or materials; and likewise the owner is more apt to give favorable consideration to the contractor's suggestions and to offer his co-operation and assistance in such matters as steam, light, power, railroad siding, supplies, purchasing, housing, and the like. Under a lump-sum contract, if the contractor comes to the owner with suggestions or requests, the owner is liable to say, "You have your lump-sum contract. Why should I help you out? I have no more interest than to see that I get what I want."

If, as is the case in nineteen out of twenty building contracts, changes, alterations, or additions are made in the work to be done after the contract is signed, the owner is at the mercy of the contractor on a lump-sum contract. We all know how difficult it is to put a contractor off the job, and how unusual it is to get another contractor to come in and do extra work when the first contractor is still on the job.

A friend of the speaker, a distinguished New England contractor, in the same line of business, puts this whole question in another, but particularly practical, way. His statement, as the speaker recalls it, was to the effect that he would like to see a comparison or a statement of the difference between the total cost paid by the owner for a given piece of work under a lump-sum contract when all extras had been paid for, and the cost of a similar piece of work done on a straight cost plus fee contract. It was his judgment that the average lump-sum contract plus extras, would cost the owner more than the "cost plus" or service form of contract.

The speaker would like to say just one word as to the recent vigorous condemnation of "cost plus" contracts. The question would seem to be not so much one of different types of contract as one of conditions under which work was done. It is hard to see any valid reason why labor should be less efficient on a percentage job than on a lump-sum job. A workman is naturally primarily interested in his own welfare and, therefore, in the keeping of his job. If negligence and a lack of interest in his work lead to his discharge on a percentage job just as quickly as on the lump-sum job, while he finds that diligence on the percentage job is awarded by advancement and continuous employment by the contractor, he will work just as hard on one form of contract as on another. The whole thing really comes down to the disposition, intent, and organization of the contractor.

There has been a pretty wide disposition to discredit "cost plus" contractors for the Government during the war and to blame the high costs of Army work on the "cost plus" contract. The speaker feels

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that this is wrong. In the majority of cases the jobs were in out-of-the-way places, where it was extremely difficult to obtain labor, also at a time when there was a great demand for labor. Furthermore, the speaker feels that the war costs, high as they are known to be, resulted not only from the conditions mentioned, but also because of the inexperience of the workmen. The experienced workman in most lines of industry continued to give good services during the war when working for his old employer, but there were not nearly enough of these experienced workmen to carry on the Government building programme, and it was necessary, therefore, to employ large numbers of men new to the industry. These men were frequently found to be inefficient.

At present, the speaker's Company is erecting more than 50 buildings, practically all on a "cost plus" basis, and generally apparently to the owners' satisfaction. The owners on more than 200 other buildings, which the Company has built on "cost plus" contracts, seem to be satisfied with the costs, time, and quality of the work which they got. This is indicated by the surprisingly large number of repeat orders for buildings from "cost plus" clients. As a matter of fact, of a total volume of nearly $50 000 000 of private business, 72.2% has been on a repeat-order basis. This would seem to be a pretty good answer to the objections to "cost plus" contracts for private industrial business.

The Company kept a cost system regardless of the type of contract. The jobs are divided into some fifty odd accounting units, that is, detailed cost units, very much on the basis of manufacturing shop costs. The units under discussion are the pure net units without any respect to profit, overhead, insurance, or other general job or main office top charges. The costs quoted for lump-sum contracts are the net costs to the contractor. The contract price does not enter into the question. It is just as though one took the shop cost of making a wheel. This cost of that wheel would be presumably without any overhead expense; it would be merely the labor that went into the wheel, as distinct from the materials or the selling price. The Company sought to arrive at the unit costs.

To do building work economically, expeditiously, and well, in reinforced concrete, requires skill and complete organization and experience. Only contractors who can get work on a reasonable basis of compensation can retain organizations of this type. Many contractors can go in and gamble on their profits on a "hire-and-fire" basis, but the conservatively organized concern prefers not to take a chance on losing money, but to take only such work as gives a reasonable profit.

The figures quoted do not include the contractor's profit. They are merely unit costs or shop costs aside from insurance. The results, although for 1917 only, check very closely with 17 years' experience

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on more than 500 buildings. The Company has known for years that it got lower costs for the owner under a "cost plus" contract than it could for its own pocket-book on a lump-sum contract.

In reply to the question as to what is meant by "cost plus," whether it is cost plus percentage or cost plus a lump sum, the speaker would say that the 22 buildings under discussion were about equally divided, on the one hand, between the cost plus straight percentage on a sliding scale where the contractor's compensation varied directly as the cost of the work, and, on the other hand, a cost plus a fixed fee or lump sum, where the contractor's compensation was independent of the cost of the work. Some of these contracts had guaranteed totals with or without profit-sharing. There are many different ways of writing "cost plus" contracts; any one can sit down and work out twenty or more of them in an afternoon, but it is a fact that the best units, that is, the best net units of the cost of the work, have been made on unguaranteed jobs, and the reason for this is the spirit of confidence and co-operation which usually cannot be gotten from the owner under a lump-sum form of contract.

In regard to whether or not the lower cost of the "cost plus" contract in the speaker's organization was reflected in the amount that the owner had to pay for the job, or whether the Company made more money out of it, the speaker would say that if the contract was a straight unguaranteed cost plus percentage form, the cost paid by the owner is the net cost plus the fee, so that any variation in cost benefits the owner directly. If the contract has a guaranty or upset price, it usually also requires a profit-sharing clause, so that any lowering of unit net costs would benefit both the owner and the contractor. This profit sharing varies from one-half to one-third to the contractor and two-thirds to the owner. Occasionally, the owner's share is even higher.

On some of the Company's contracts it has profit-sharing, but not on a lump-sum contract. In this latter type, the contractor makes all the profits on any unexpended allowance that there may be. A "cost plus" contract with a guaranteed total is really an attempt on the owner's part to "eat his cake and keep it too." It is a cumbersome contract, because it is subject to all the careful estimating and bargaining on extras and credits that the lump-sum contract effects, and, at the same time, requires the minute cost keeping and negotiations with the owner which a "cost plus" job involves.

On every job the Company has, its superintendents' profit-sharing plan applies. In explanation of that plan, it may be stated that ever since 1910, the Company has been saying to the members of its construction organizations that if they equal or come inside of an agreed estimate, the superintendent, and on big jobs, his chief assistants, will get extra compensation or bonuses, varying by the amount that the estimate is exceeded. Recently, this plan has been extended to include the

principal foremen on every job. The speaker has mentioned an agreed estimate, and by this, he means that, on cost plus fee jobs, the estimate is submitted to the architect or engineer and the owner for their approval as a reasonable amount to allow for the work in the immediate control of the job organization. On lump-sum contracts, the estimate is not submitted to the architect, engineer, or owner. It has, however, on either kind of contract, to pass the profit-sharing committee of the Company. Mr. Perry.

On "cost plus" work the owner is told that any bonus earned under this plan is part of the cost of the work payable by him, because such bonus or extra compensation is, in a measure, extra salaries paid to the organization in order to get lower costs, and the owner benefits equally.

The statement has been frequently made that on "cost plus" work the men actually doing the work in the field know the character of contract, and the pocket of the disbursing agent, be it the Government or a private owner, suffers accordingly. The experience of the speaker's Company contradicts this, and it is believed that that experience has been sufficiently wide to make its opinion of fair value. As stated, the whole question is one of the spirit of the organization. If it is known throughout the rank and file of a contracting organization that a man is discharged just as quickly for careless shirking work or inefficiency on a "cost plus" job as he is on a lump-sum job, experience indicates that no difference can be detected in the unit costs between a "cost plus" or a lump-sum job. Unit costs really tell the story of the relative operation of the two contracts.

If there is efficient management, systematized inspection, and a determination to see that an organization is run on an even keel, regardless of the form of the contract, any difference in the unit cost on a job due to the form of contract would be in general in favor of the "cost plus" job.

Relative to the closeness of the two forms of contract in their comparative costs being due to the fact that the superintendents and the foremen shared in the profits, the speaker thinks that undoubtedly that is true, but it seems to him that if this point is admitted, it might be also admitted that the form of contract really makes no difference at all. The speaker recently made up some figures showing the difference between the estimates given by his Company to owners at the time of negotiating the contracts for their work and the actual costs which the same owners paid for the completed undertaking. These figures cover all "cost plus" contracts made between July, 1910, and January 1st, 1918, that is, about 7½ years, and cover about $12 500 000 worth of work. The actual cost paid by the owners, including all profits to the contractor and any bonuses to field organizations of the contractor, were less than the estimated costs by about

Mr. Perry. $400 000, or approximately 3 per cent. This covers a total of about 90 contracts for probably 115 buildings.

It would seem that this was a pretty good indication of the excellence of the management, the ability to estimate correctly the costs, and the consistent performance of the field organizations in carrying out the schedules set up for them. It checks back again to the proposition that the type of contract is not the determinate factor. The really important feature is the management and, perhaps, the intent and disposition of the contractor.

Mr. Seaman. HENRY B. SEAMAN,* M. AM. SOC. C. E.—In the discussion of this paper, reference has been made to Government work or, more properly speaking, to war work, and the apparently excessive cost of the "cost plus" contracts for the Government during the war. The cost of war work should never be compared with similar conditions on private work, as the cost of work for war purposes was of minor importance when compared with the time to be saved in its completion. There was an "enemy at the door", and an extra cost of, say, $1 000 000, on a large piece of work, was not to be compared with the possible saving of 2 weeks' time in its completion; not only were many lives in jeopardy, but victory itself was at stake.

At the beginning of the war, there was no time to enter into formal contracts. Contracts amounting to millions of dollars were awarded verbally, the terms to be confirmed in writing later, and the increasing number of contracts continually awarded gave no opportunity for details.

In due time, the various forms of contracts were fully considered by a board representing six different branches of the Government, and although the cost plus percentage form was considered the least desirable, it was found, for a considerable time, to be the only practicable form which could be adopted under the stress of circumstances. Later, however, modifications and improvements were introduced gradually and, in many instances, a bonus was offered to the contractor on any saving, under an agreed estimated cost.

The successful execution of a "cost plus" contract, whether on percentage or fixed fee, depends largely on the character of the chief contractor and his ability to control the execution of the work. Almost invariably his character will be found to pervade the entire work. It is for this reason that percentage work must be awarded by personal selection rather than by competition, and as personal selection may have the appearance of favoritism, it will be difficult to establish the practice on municipal or other public works.

* New York City.

Mr. Glaeser.

FREDERICK GLAESER,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer is strongly imbued with the idea that the functions of the engineer do not end with the preparation of a set of specifications and drawings for a particular piece of work, but, like those of the sculptor, must include its completion, as created in his brain, even to the last detail of construction; he must not place on a second party, called a contractor, the responsibility for carrying out the plan. The thought which the writer desires to convey is that the middleman, as in merchandising, is a source of expense that possibly may be done away with in the future. This thought is highly important in these days of high prices, when expenses are pyramiding one on top of another, and the greatest opportunity for the engineer to do constructive service to-day lies, it is believed, in developing his executive powers, in order that he may be able to carry out properly a pre-arranged construction programme.

We all credit the contractor with the "know how", and, being in business to secure his living, he must earn it; but, because of the gamble that he must invariably take, he is bound to protect himself so that he will not lose. The natural thing, therefore, for him to do is, of course, to bid sufficiently safe so as not to pay for the privilege of doing the work. We have all had experience with various classes and types of men, and know that some must be watched more closely than others.

During the last two years, the writer has had opportunities to judge of the "cost plus" contract in its various phases, and is entirely in accord with, and a strong supporter of, the type. The forms used have included the "percentage fee" on a sliding scale; the "fixed fee" form, in which the fee was agreed on by both parties and usually was a certain percentage of the estimated cost agreed on by the owner and contractor before the work was begun; and, latterly, on small jobs, due to the fact that surplus materials were available, and because materials could be purchased at Government prices, the policy has been to assemble all the materials at a convenient storehouse or storage pile and ask for bids on furnishing the labor and equipment required to perform the work. This form of contract is limited in its application, necessarily, but has the advantage of employing the man who possesses the knowledge and experience, but lacks the capital. It has been proven to result in considerable economy. On work valued at approximately $100 000, made up of seventeen jobs, it has been possible, with a gang of general average characteristics, to save a considerable sum.

If it is necessary to enter into a contract, the ideal form, in the writer's opinion, and that which permits of more elasticity in its operation than those at present in vogue, is one in which the estimated

* Trenton, N. J.

Mr. Glaeser.

cost is agreed on, the fee fixed at a certain sum subject to adjustment in one of two ways, namely, a bonus if the actual cost is less than the estimated cost, and a penalty if the actual cost exceeds the estimated cost. It will be admitted that changes in plans may change considerably the cost of the work, but in that case the ingenuity and knowledge of the engineer is brought into play, adjustments are made on the basis of the original contract, and additions or subtractions are made from the sums involved. The tendency of this form of contract to make the parties thereto feel that a partnership exists, which can be beneficial to both, is an incentive with unlimited possibilities, and results in economy not otherwise obtainable.

In continuation of the idea expressed in one of the foregoing paragraphs, it would seem that one of the functions of a National Engineering Society would be the proper supervision of the execution of Federal, State, and Municipal undertakings of an engineering nature. It would seem that such a society, through its various local associations, could be so in touch with civic improvements as to bring about a considerable reduction in the cost of engineering works. This could be done by providing a clearing house through which the various services necessary for any piece of work could be supplied. Tools and equipment could be purchased or hired, and the middleman eliminated. Such a combination would give the engineer the opportunity to lessen the burden on the tax-payer, to reduce the cost of living very greatly, and to give a maximum of service to the community.

Mr. Snyder.

GEORGE D. SNYDER,* M. AM. SOC. C. E. (by letter).—There is one method of constructing engineering work that was not referred to in the paper and was only mentioned incidentally in the discussion, that is, where the engineer executes the work without the medium of a contractor, by employing labor and purchasing material, either in his own name or that of the owner, thereby becoming the owner's agent. By this means, instead of having three organizations to deal with—the owner's, the engineer's, and the contractor's—there are only two, the owner's and the engineer's.

When an engineer is retained to design and execute an engineering work, and the owner engages a contractor to do the actual construction, what he is really doing is employing a man who is skilled and experienced in the business of constructing to execute the plans and specifications of the engineer, whose remuneration is fixed in the contract whether it be a lump sum, by unit prices, or some form of "cost plus". In any case, he must produce a cheaper or better structure, or he must produce it in less time than it could be done without him, in order to justify his existence. In other words, he must effect sufficient saving to recompense himself, and, in addition, effect a saving

* New York City.

to the owner in money or time. As a matter of fact, on most works of the usual character, under stable conditions of prices of labor and material, the contractor does justify his existence. Mr. Snyder.

No general rule can be laid down as to which type of contract is preferable, as each has advantages under certain conditions.

The lump-sum form of contract is preferable where the character of the work is not unusual and where the quantities can be determined accurately in advance. It has the advantage to the owner of enabling him to figure with some accuracy, even allowing for a certain percentage of extras, on his probable capital outlay.

The unit price contract is preferable where the quantities cannot be determined accurately in advance.

The "cost plus" contract is more adapted to work where there is not sufficient time to work out full plans and specifications in advance, or where complete data as to the conditions to be met cannot be obtained.

The conditions adaptable to the work being done directly by the engineer are as follows, some of which however apply with equal force to the "cost plus" method:

1st.—Works of unprecedented character and difficulty.

2d.—Works where the time element is the ruling consideration and where there is not sufficient time available to work out completely and develop the plans and specifications before starting the work.

3d.—Works likely to extend over a long term of years during which marked changes in the prices of labor and materials and in the conditions under which the project is financed, may occur.

4th.—Works on which safety is the dominating consideration, where changes and alterations are to be made in a structure without discontinuing its use by the public.

Among works of unprecedented character and difficulty may be mentioned the completion of the Hudson River Tunnels of the Hudson and Manhattan Railroad. At the time work was resumed on these tunnels, the project had a bad financial history. The company had been in the hands of a receiver, and several contractors had been unable to complete the work. Therefore, when the owners invited bids to complete the project, such bids were so high that it was decided to reject them and execute the work at cost under the direction of the engineer. This was so successful that the entire system, with the exception of portions of the work on the land approaches, was done in this way.

The Hudson River Tunnels are also an illustration of the advantage of using this method where the time element is the ruling consideration, for if no work had been done until complete plans and specifications had been worked out, it would have delayed the ultimate com-

Mr. Snyder.

pletion very much. As it was, when the tunnels were partly completed, the management changed the plans in order to permit the use of multiple-unit trains instead of single-unit trolley cars, which was done without discontinuing any of the work in hand.

In regard to works extending over a long term of years, two instances may be mentioned, one having been done at cost by the engineer and the other by an ordinary unit-price contract. The financial panic of 1907 involved both owners so that they could not continue the work on account of their inability to get funds to meet bills and pay-rolls. In the first case, the construction of portions of the work was suspended without any legal or other difficulty, while, in the second case, a settlement had to be made with the contractor to compensate him for his loss due to the suspension of the work.

A case involving safety may be mentioned, namely, the alterations to the Center Street Loop to conform to the plans for the Dual Subway System after two of the tracks were in operation. In this case, a contract was let to the operating company to do the work at cost, and the work was executed under the direction of its engineers. It would have been very difficult to have drawn a contract that would have safeguarded the public and, at the same time, properly recompensed the contractor for the extra work necessary to do this under any other form of contract.

In doing work at cost, under the direction of the engineer, it is essential that he shall have the necessary experience and organization and that the necessary arrangements in regard to plant can be made. In cases where the plant is special, or where it will not have a longer life than that necessary to complete the work under consideration, it must be purchased and charged off to the work regardless of the character of the contract. Where the plant is very expensive in proportion to the cost of the work, and cannot be leased, it may be necessary to let the work to a contractor having a suitable plant.

To claim that the same efficiency of labor cannot be obtained on a cost job as on a contract job is an admission of inefficiency on the part of those directing the work. Of course, it is human nature for a contractor who has both contract and force account jobs under way, to assign his best men to the contract job; but there is no such incentive where the work is being done by the engineer directly as agent for the owner.

Doing work by the engineer without the medium of a contractor is applicable to public work, where State laws do not preclude such a method, but trouble may be had with labor owing to civil service requirements and rulings. Some of the Federal river and harbor work is executed by direct labor.

The advantages and disadvantages of these methods vary with the character of the work, the local and financial conditions, and the

owner's desires, and the engineer should not limit himself to any one method of executing his plans, but should freely use those best meeting the conditions. Mr. Snyder.

F. G. Baum,* M. Am. Soc. C. E. (by letter).—The writer thinks that the "cost plus", or percentage, plan of doing construction work is an abomination and a practice which leads almost inevitably to loss of control of the cost of the work and loss of confidence of the owner in the engineer and contractor. Mr. Baum.

The contractor or engineer may be ever so honest, but the moment the men in his employ know the job is a percentage job, they think they are being loyal to their employer if they make all the "percentage" they can. This applies from the top down to the last laborer. The writer has heard the men (big and little) brag about how they "put it over" for the boss. Until human nature changes, the "cost plus" plan must lead to great dishonesty. The author admits:

"There can be, of course, great dishonesty under 'cost plus' contracts, but, if this occurs, it is the owner's fault in lax supervision of his own forces or from incompetent or dishonest engineers."

That is a remarkable statement. What control has the owner over the contractor's dishonesty or that of his men, or their competency?

The writer has always made it a practice to have nothing to do with the "cost plus" plan, except where no other course could be followed; and he thinks engineers, contractors, and architects, generally, will obtain a higher standing, if they abandon it entirely, except in emergency cases.

George Perrine,† M. Am. Soc. C. E. (by letter).—"A contract is an agreement between two or more parties for the doing or the not doing of some definite thing."‡ Mr. Perrine.

In construction contracts the contractor agrees to do certain work and furnish the necessary materials for stated prices, for which he is to be paid by the owner. The prices are always stated in the contract, and the units for which these prices are to be paid can be accurately measured or counted.

For doing certain work described in the specifications, including the furnishing of all materials enumerated in the contract, the contractor expects just compensation (in addition to actual costs) for his services and experience, and the use of his organization, plant, and money in prosecuting the work.

The owner knows what the completed work will be worth to him; if it is a factory, it will give certain returns in rent, and so with other things, a value can be set on them. In order to fix the value, the owner

* San Francisco, Cal.

† New York City.

‡ "Contracts," by Parsons, Vol. I, p. 6.

Mr. Perrine.

engages an engineer or architect to make the estimates. If he is a good business man, he will engage an experienced adviser, as an insurance against serious mistakes in fixing the the value of his new property.

The owner has the estimate of his expert from which to judge the amount of capital involved; then he advertises for bids. Usually, such bids are called for as unit price or lump sum; in either case, the bid prices are to include the cost of the contractor's services and his profit. A number of contractors will bid, their bids varying more or less from the figures of the owner's expert. The expert can judge very closely from experience by comparison with the contractor's bids how much the contractor will probably realize for his services.

The contractor's first thought, when making up his bid, is: How much money has the owner appropriated to do the work? The next is: Who are to be my competitors? Then he bets a certain amount on his skill against that of the other contractors.

If the contractor's figures are much lower than those of the owner, the supposition is that he will lose money if awarded the contract and, therefore, trouble and disputes will arise between the contractor and the owner, which may be more expensive in the end than if a contract had been let to a reliable contractor at the owner's figures.

There are certain things for which the actual costs can be fixed before commencing construction. Quotations can be secured, good for a stated time, for materials and plant rental, these cost prices to hold good while a contractor is being selected.

After these provisional agreements are made, and a contractor is found who will be willing to do the work at the owner's estimated cost, the owner can decide whether or not he wants to build at the prevailing market prices. He then has his expert estimate the probable cost and fluctuation of the prices of labor, and the cost of labor and materials are added together to obtain the probable net cost of the work.

The owner knows what he can afford to pay for what he expects to get under the terms of the contract, but is often uncertain as to the personal ability and integrity of some of the contractors and their forces. If a contractor is losing money, the owner will probably have some difficulty and be put to additional expense to guard against getting poor material and workmanship.

It should be possible to guard against such conditions by letting a contract to one of several selected firms, the contract to cover the cost of the work based on the owner's estimate for the actual cost, including overhead expenses and the contractor's fee. The contractors who are invited to bid for the work should be required to divide their bids into three parts, net cost, overhead, and fee desired.

In order to give the contractor an incentive to save money, he should share with the owner any saving below the latter's estimated cost.

Mr. Perrine.

It is the duty of the owner's engineer to see that the contractor does not turn out poor work, in order to make a saving and, thereby, get an extra profit. It is also his duty to see that his inspectors' methods do not retard the contractor's forces.

The owner's engineer should add to his net cost the contractor's overhead charges and fee and a certain amount for contingencies. When the contractor signs the contract, he accepts the approved estimate of cost and contingencies as final and binding.

The contractors under consideration by the owner should be requested to examine the plans and make their own estimates, in order to decide whether they are willing to accept the contract based on the engineer's estimate.

If, in figuring the contractor's fee, the total cost of the contract should exceed the net cost plus, say, for example, 50% of the estimated contingencies, a deduction should be made in the contractor's percentage fee, by an amount in proportion to that which the cost exceeds the net cost, plus 50% of the contingencies. The object of this is to compel the contractor to exercise the best supervision in order to keep down the costs. By such a contract, the owner pays no more than his engineer estimates the cost to be, and the contractor has the opportunity of increasing his profit by exercising his skill. Orders for extras may be figured on the same basis as those for the general contract.

Mr. Reed.

P. L. Reed,* M. Am. Soc. C. E. (by letter).—When a prospective owner has decided to apply his funds to the construction of some structure or other engineering work, he may handle it in one or several ways:

(*a*) Employ the necessary foremen, mechanics, and laborers; purchase the necessary materials; and supervise the construction.

(*b*) Employ one or more superintendents on a salary basis and delegate to them the authority to hire some or all foremen, mechanics, and laborers; and to purchase some or all of the materials; and entrust to these superintendents the application of the labor and the materials.

(*c*) Employ a contractor on a "cost plus" basis and delegate to him most or all of the authority to employ labor, purchase materials, and apply them to the work to be accomplished.

(*d*) Enter into a fixed price or fixed unit price contract by which he delegates all authority for employment of labor, purchase of materials, and their respective applications.

Method (*a*) is "day labor" or "force account"; Method (*c*) is "cost plus"; Method (*b*) is an intermediate stage between Methods (*a*) and (*c*); and Method (*d*) is "contract" work.

* Washington, D. C.

Mr. Reed.

It is to be noted that Methods (*a*), (*b*), (*c*), and (*d*) involve increasing delegation of authority, but that in Methods (*a*), (*b*), and (*c*), the owner retains the financial responsibility for results, and that only in Method (*d*) is he relieved of this. As Methods (*a*), (*b*), and (*c*) involve increasing delegation of power to spend without definite responsibility for costs, they imply increasing degrees of confidence or trust by the owner in his agents, coupled with financial supervision in different proportions according to circumstances. This element of trust which attains maximum proportions in the "cost plus" method of doing work is of so intangible a nature as to put this method under a great handicap in the award of public works, in normal times, and where competition is keen, even when permitted by law.

The author has suggested that "cost plus" contracts could be let on the basis of fee demanded, on the contractor's resources, and on his reputation. The only really tangible item in such a valuation of a competitive proposal would be the amount of the fee demanded, and to attach much weight to this item would be like selecting a doctor, a lawyer, or an engineer on a similar basis. The vital question is how well will he perform. The really important items are the intangible ones, and it must always be difficult, if not impossible, in normal times, to justify to the satisfaction of public bodies, awards of public works on such considerations.

The author explains the difficulty arising under fixed price contracts from extras, actual or alleged, which are largely avoided under "cost plus" contracts. On the other hand, the "cost plus" contract brings its own difficulties in the general control and auditing of the contractor's expenditures and in deciding what items are properly to be included in the cost of the work and what items are chargeable to the contractor's account, particularly as to items of indirect or overhead expense. Both kinds of difficulties, of course, are minimized by definite and complete plans and specifications in the case of the fixed price contract and a definite and comprehensive contract in the case of the "cost plus" contract.

In his advocacy of the "cost plus" method, the author suggests that the gamble belongs to the owner and mentions the uncertainties of site, weather, and varying market conditions; but what should be said about the gamble of competent and honest supervision and direction of purchases, plant, construction schedules, and organization and application of working forces? This latter part of the risk, and it is an exceedingly important part, belongs as logically to the contractor. If it be urged that the owner, himself, may supervise these things, then the contractor becomes little more than a salaried superintendent and we approach "day work" or "force account".

However, the gamble may be conceived to be divided in origin. It is within the power, and it is the practice, of the two contracting

parties to distribute it as they may agree. In some cases, the owner may be in a better position to assume the risks of the enterprise, in some cases the contractor; but, independent of this, one party may be more willing to assume risks than the other, and this will have a material effect on the form of the contract drawn to accomplish the "meeting of the minds". Mr. Reed.

The facilities of modern trade enable one to take risks or transfer them to others within a large range. A miller may buy his grain on the Exchange before it is harvested, for a definite price per bushel for future delivery, and we are told that various crops, such as fruit, are not infrequently sold by the producer for a definite lump sum on the trees, before they are ripe. We may carry insurance of many kinds to relieve ourselves to a large extent of various risks. An owner, either an individual, a private corporation, or a city, may have a sound reason for placing a contract for some improvement at a definite lump sum price, with the contractor assuming all risks. Nor does it follow that a fixed price contractor in assuming risks charges, in the long run, for contingencies which do not happen and that, on the whole, the owner pays more than he would had he assumed the risks. The contractor would naturally prefer to cover in his fixed price all possible contingencies, but if he did he would get little or no work in competition, and it is doubtful if, as a whole, contractors, in their estimates, do cover the contingencies which actually develop.

In manufacturing, the man in the shop may be paid by the hour or by the piece, with or without bonuses, premiums, or other elaborations. Both basic methods have been in use for many years, and there is yet no consensus of opinion as to which is the better. It is probable that each is to be preferred under certain conditions. Similarly, in contracting, the contractor, like the day worker, may be compensated by a fee, regardless of efficiency of performance, under a "cost plus" contract; or, like the piece worker, he may be paid a fixed price for accomplishing a definite result. In this case, too, the method to be preferred probably depends on the circumstances.

Recognizing that in the maximum delegation of authority to spend without definite responsibility for costs and incentive to economy lies the inherent defect in the "cost plus" contract, various modifications have been made in these contract forms.

The general types may be indicated as follows: (*a*) cost plus a percentage fee; (*b*) cost plus a percentage fee with a fixed maximum cost; (*c*) cost plus a percentage fee with a maximum fee; (*d*) cost plus a fixed fee, based on the estimated cost of the work; (*e*) cost plus a fixed fee with a bonus represented by a definite proportion of any savings effected in actual cost below the estimated cost. A limit may be set to the amount of the bonus; (*f*) cost plus a fixed fee with the bonus for saving as in (*e*) and a penalty repre-

Mr. Reed. sented by a definite proportion of any expenditure in excess of the estimated cost. A limit may be set to the amount of the penalty so as to leave a minimum fee.

It will be evident on consideration that Modifications (*b*), (*e*), and (*f*) of the simple "cost plus" forms are essentially compromises between the straight "cost plus" and the fixed price forms, and that they serve to divide the risks which the owner carries in a straight "cost plus" contract and which the contractor carries in a fixed price contract.

The exceptional conditions produced by the war had as elements: (*a*) The importance of a prompt commencement and vigorous prosecution of large and complicated projects of indefinite extent and detail, with economy a distinctly secondary consideration; (*b*) large, rapid, and uncertain fluctuations in prices of material and wages of labor; (*c*) serious disturbances in the normal operation of the law of supply and demand; (*d*) an exceptional degree of patriotic feeling which minimized fraud and induced impersonal devotion to the work.

These conditions were favorable to the "cost plus" method in the execution of public works. When a contractor is given a "cost plus" contract, he is essentially appointed to a position of trust, and the general power of committing the Government or a city to this relation is not one which the public will lightly place in the hands of its contracting officers in normal times.

Mr. Wheeler. WALTER H. WHEELER,* ASSOC. M. AM. SOC. C. E. (by letter).—The author sets forth in his paper certain advantages claimed for the "cost plus" contract over the "lump sum" or "item" form of contract, and invites discussion of the question.

There is little doubt that the majority of contractors favor "cost plus" contracts. On the other hand, it is the writer's belief that very few owners favor this form of contract, and there are certain good reasons why they should not, except on certain classes of work, such as repair work and excavation and grading where the kinds and quantities of materials are difficult to determine. The writer has been fortunate in his experience in being at various times a principal on construction in the capacity of engineer, contractor, and owner, and thus has had opportunity to observe and study contracts from these different viewpoints.

It seems clear that most of the advantages claimed for the "cost plus" contract vanish completely for most construction work, when the engineer who draws up the plans and specifications, performs his duty thoroughly, conscientiously, and intelligently.

One of the stock arguments which is often advanced by contractors in favor of the "cost plus" contract, is the time to be saved by letting the contract and starting the work before the plans and

* Minneapolis, Minn.

specifications are complete. This is a very specious argument, and frequently owners are persuaded to let their contracts in this manner, cherishing the belief that time will be saved. Rarely is this expectation realized. On the contrary, the result is usually disappointing to the owner. Time usually is actually lost in straightening out the errors and mistakes that have been made in starting before everything had been worked out carefully on paper, and the cost, instead of being reduced, is increased, on account of the inability to organize the job properly for efficiency because of lack of knowledge of all the details of the work involved and through the errors previously mentioned. One reason why this argument of saving time is used so often by contractors who are after a certain piece of work, is obviously because they well know that after all the plans and specifications have been worked out, they will not have convincing arguments to present in favor of the "cost plus" contract. Certainly, no greater mistake can be made than to begin any undertaking, whether construction or other enterprise, without proper preparation, and proper preparation in a construction enterprise certainly involves complete plans and specifications. Mr. Wheeler.

In the absence of complete plans and specifications, the basis for a clean-cut contract is lacking. Under certain circumstances, haste in beginning work is sometimes justified, such as the necessity of retaining right-of-way rights, title, or some similar reason. However, it is not necessary as a rule to let a contract for more than a small part of the work in order to accomplish such a purpose, and the plans and specifications for the major work can be effected while the contracted portion is in progress.

It is the duty of the engineer to advise and even insist that his client have enough exploration work done to furnish reliable data on which to base his plans. Such work is usually relatively inexpensive and will frequently result in an entirely different plan being used. Often, in this way, many times the cost of the exploration work is saved. Engineering is or should be an exact science, and unless one has exact data on which to base plans and specifications, one is working on guesswork and not on fact.

It is argued that "cost plus" contracts save money for the owner. It is possible that such results are sometimes attained, or could be if plans and specifications were worked out with the same care and completeness under this system as they would be under the "lump sum" system. Here, in the writer's opinion, lies one of the dangers of the "cost plus" contract to the Engineering Profession. Regardless of how conscientious the engineer may be, he is almost certain to be less thorough in his work when he knows that there will be no "comeback" at him on account of extras resulting from his failure to have his plans and specifications complete, because it will simply be a

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matter of correcting the error or supplying the deficiency when discovered; but the cost is there just the same, although it may not appear as an "extra". The writer is very much inclined to be skeptical as to the saving that can be made on the cost.

Another argument advanced for the "cost plus" contract is the ease with which changes can be made as the work progresses, and one might get the impression from reading the paper that since these changes are easily or more easily made under the "cost plus" contract, they involve no additional cost because of the change. On the contrary, every contractor knows what it means to be making frequent changes in plans after work is started, or, worse yet, to be correcting or changing work which is already in place, and the greatly increased cost that results over what such cost would have been had the work been done that way in the beginning. Here, again, is another pitfall for owner and engineer. When the changes have to be taken up and their cost arrived at before they are made, as under the lump sum contract, the owner is impressed with such cost. Under the "cost plus" contract, the owner often does not awaken to a realization of the cost until the job is done and the final cost is known, and, in many instances, the job has not been finished, because adequate funds were not provided in the beginning to cover the total cost; or, if it is finished, it is a disappointment from an investment standpoint, as all the preliminary calculations are upset. Here, again, a heavy responsibility rests on the engineer to see that his client, and also that he, himself, studies the work so thoroughly and considers every detail so carefully while the plans are in preparation, that it will not be necessary to make changes after the contracts have been signed.

The writer cannot agree with Mr. Clarke's assertion that the "lump sum" contract has not the advantage of showing the cost of the work before starting, and that there is "invariably a long list of extras." If there is this "long list of extras", the writer would be inclined to place at least a part of the blame on the engineer who prepared the plans and specifications. He knows of considerable important work that has been carried through without, or practically without, a single extra. However, he sees no good reason why extras under the "lump sum" form of contract should be so difficult, provided a schedule of prices for extra work or deductions to cover contingencies is required of the contractor when the contracts are signed.

It is vitally important to the owner that he know very closely what the work will cost him before he awards the construction contract. There are many instances where owners have relied on the estimates of engineers or architects for the cost of the work or on the estimates made by contractors based on preliminary plans or sketches, with disastrous results. There are even instances where unsuspecting owners have been encouraged to go ahead with work,

usually on the "cost plus" system, by willfully misleading estimates, and, in some instances, have met with financial ruin as a result. In the writer's opinion, the engineer, architect, or contractor who so misleads an owner, either knowingly or through ignorance, should be severely punished under the law, and certainly this Society should not countenance anything of the kind and would not if the writer correctly understands its ideals. Mr. Wheeler.

The same principles will hold good on the argument that better workmanship is secured under the "cost plus" contract. It is simply a matter of making the plans and specifications clear and definite and seeing that they are followed, which would be just as necessary under the "cost plus" system. The writer can see no reason why the contractor bidding on a lump sum contract should have to "guess at" the meaning of the specifications. He has the privilege of making definite, by correspondence with the engineer, anything that is indefinite or difficult to interpret. If the plans and specifications are well drawn, why should there be anything in them that is indefinite? Is it not for the engineer to see that his plans and specifications are intelligible?

It would seem that for most work the "lump sum" form of contract comes nearest to meeting all the conditions and requirements of construction work in the best manner, and it is a matter of the engineer's ability, intelligence, and integrity to make it work out smoothly, fairly, and properly.

It is only fair to the contractor that he be protected against increases in wages, freight rates, and other contingencies such as strikes. As to material prices, he can secure protection for a long enough time to enable him to place his orders, and if he is negligent that is his lookout.

It seems to the writer that for public work the "lump sum" is much to be preferred to the "cost plus", as there are already too many opportunities for politics and "wire pulling" and these, in the writer's belief, would be increased by the adoption of the "cost plus" contract.

E. G. Walker,* M. Am. Soc. C. E. (by letter).—An examination of this paper leaves the impression that its author is a strong advocate of the "cost plus" form of contract, almost to the exclusion of other forms. In his strong advocacy of it, he enumerates a number of points which he reads as great advantages of the system, but which to many other people would appeal rather as disadvantages. The whole question as to the best form of contract to be used for a particular job turns entirely on the circumstances under which the contract is being issued, and it is practically impossible to say that one form is better than another unless the particular circumstances of each case are taken fully into consideration. Mr. Walker.

* London, England.

Mr. Walker.

In general, in engineering practice, there are three parties primarily involved: (1) the owner; (2) the engineer; and (3) the contractor. The second and third parties have their status fairly closely defined, in that they are individuals or firms who specialize in their particular branches and are bound by the laws and customs of their respective professions. The owner, on the other hand, is drawn from the widest possible limits and may be a unit of any size, from a single individual to the whole nation. Further, he may be technical or non-technical, and the work which he is having done may or may not be embarked on for profit.

A still more important feature is that the nature of the work to be undertaken is in some circumstances definitely known before the contract is entered into. In other cases, it may be possible at best only to obtain an approximate idea beforehand of what has to be undertaken after the job has been started. To meet all these varying conditions and circumstances requires the development of several different types of contract. At the present time, conditions are probably much more favorable to the "cost plus" system than they are likely to be in more normal times, because, owing to shortage of materials and fluctuations of wages, it is difficult in many cases to get contractors to commit themselves definitely to prices for the necessary time ahead, which they have to do if they tender on the ordinary lump-sum basis.

The impression left by the author's case for the "cost plus" system is that when this system is developed to its utmost, the owner virtually becomes his own contractor, except that he hires the plant from the regular contractor and utilizes him as a kind of extended ganger over the working force. All responsibility as to the actual carrying out of the work is apparently to be left to the owner, who is also to shoulder all trouble that occurs with labor, and all risks, and to insure remuneration to the contractor no matter how the work may turn out. This seems to be rather an inversion of the ordinary notion. The writer believes that the generally accepted idea is that a contractor is paid and allowed to take profit on work mainly by reason of the fact that he is prepared to relieve the owner from the responsibilities of carrying out the work, and in the majority of cases it is believed that the principal advantage that owners see in putting work out to contract, is that their expenditure will be known beforehand within reasonable limits, and that the responsibility for the proper carrying out of the work will be largely taken off their shoulders and put on those of reputable specialists who are more competent to carry it than they are.

A point which the writer does not understand is that the author, in his concluding paragraphs, appears to imply that the "cost plus" system is eminently desirable for municipal work, although immediately before this he admits that this system allows of great dishonesty.

Mr. Walker.

The two statements appear to the writer to be incompatible, since it is self-evident that anything which allows of the development of greater dishonesty than the old system of contracts is hardly suitable for application to public bodies, in which, by the nature of their formation, the responsibility of the individual member is limited.

The writer believes that in general the standpoint to be adopted in deciding what type of contract shall be used in any particular case should be broadly that the job should be classified as either one of two types: (*a*) straightforward jobs, that is, jobs in which the amount of work to be done can be fairly closely estimated beforehand; and (*b*) uncertain jobs, that is, those in which this cannot be done. When the supply of material and labor is normal, jobs of the first class can be priced with a reasonable degree of accuracy, and for such work, in the majority of cases, it will probably be found that the owner prefers the old system of competitive tender, because his liability is fixed within moderately close limits thereby. In this class of job, alterations and additions to the original design are only small, and their value, therefore, will be only a small percentage of the original tender. With a priced schedule, the contractor does not stand a particularly abnormal chance of making a harvest on his extras, and even should his profit on them be at a greater rate than on the main contract, the actual money value is unlikely to be great enough to allow him to make an abnormal profit on the whole contract, and it is on the whole contract rather than on a few items in it that a true comparison should be based.

The second class of job is obviously the more difficult to deal with, and the two types of contract which can best be considered in connection with it, are the itemized schedule type and the "cost plus." It is probable that, in most instances, the "cost plus" will give the better results. In certain cases, it is evident that the owner must be prepared to face considerably more risk, and that his engineer must also be prepared to take more responsibility for the issue of technical instructions and orders in connection with the job. Furthermore, as the magnitude of the risk is uncertain before the job is started, it is obvious that a contractor is not in a position to estimate fairly what the work is worth to him and, therefore, must cover himself by a higher price. With the correlating circumstances that the owner is to take more risk, and the engineer more responsibility for technical supervision, a number of the disadvantages of the "cost plus" system referred to by the author are considerably reduced in importance. The writer believes that it is to this type of job that the "cost plus" system is more applicable, and he does not believe that it is a desirable system to use indiscriminately for all types of contract, and, in particular, he is of the opinion that considerable caution should be exercised in its application to cases in which public bodies are the owners.

Mr. Waddell.

J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—The importance of the subject of this paper, to owners, contractors, and the entire American nation, cannot well be exaggerated; for, until there is reached a satisfactory compromise between owners on the one hand and contractors on the other concerning the vital questions of contract-letting and profit-sharing, the business of the country will fail to recuperate to the extent that it should at this critical period in the readjustment of all constructional activities—which activities were so fundamentally upset throughout the whole world more than five years ago by the advent of the World War. The opportunity now within easy reach of the American people to secure the bulk of the world's trade is unique; but, apparently, our leaders in diplomacy, manufacture, shipping, business, and finance are either unaware of its existence or indifferent about taking advantage of their good fortune. The Latin-American nations are now knocking at our door asking to do business with us and begging us to lend them money for the development of their as-yet-almost-virgin lands, mines, and water powers, and their other more or less embryonic resources; and the leading peoples of Asia, Africa, Australasia, and even Europe are to-day much more willing to enter into business relations with this country than they have ever been before. They all recognize that, for the present at least, the European countries are in no condition to lend money, being themselves borrowers and having so many demands at home for their time, attention, and capital that they are unable to pay attention to the needs of any nation but their own; and the entire civilized world is well aware of the fact that, in spite of our vast war debt, we are to-day the most wealthy of all peoples.

These conditions—unfortunately, as far as we are concerned—will not last indefinitely; and if we are so short-sighted as to neglect to seize the golden opportunity which is not only easily within our reach, but which is actually being tendered to us and almost forced on our acceptance, it will not be long before both England and Germany will re-secure the grip on the world's trade which they possessed in ante-bellum days.

Prevention of Progress.—One of the greatest stumbling blocks in the pathway of our nation's advancement is the conflict between labor and capital; and until it is removed the wheels of progress will be clogged, and the present paralyzation of all great peace industries will continue to exist—possibly in even worse form than it does to-day. Again, the main issues between labor and capital are these questions of contract-letting and profit-sharing. If they were once settled to the satisfaction of all concerned—bankers, manufacturers, contractors, and workmen—all other minor differences would quickly be adjusted. Such a settlement is perfectly feasible; and the possibility of its

* New York City.

speedy accomplishment is neither a Utopian conception nor an idle dream. Mr. Waddell.

Primarily, the men who do the work must have a substantial share in all net profits of manufacture and construction; but the way for them to obtain it is not by the organization of strikes nor by seizing the plant and running the business of the manufacturer or the contractor. The uneducated workman is no more fit to manage business and to handle industry than the sedentary office man is to undertake the physical labor of the workman—in fact, much less so; because it would generally be totally impracticable to educate the illiterate workman up to a state of efficiency which would enable him to undertake business management and finance, while, in most cases, in a comparatively short time, the office man's muscles could be developed sufficiently to enable him to endure the physical stress of the workman's job. Nothing of value can be accomplished by mob rule, as the pending subsidence of the present wave of Bolshevism will soon prove.

Labor and Capital.—Although it is certainly true that the laborer cannot succeed independently of the business man, it is equally true that the latter cannot accomplish much without the aid of the capitalist, hence it behooves business men and financiers to come quickly to a friendly understanding and agreement. In times past, the capitalist secured and exercised a strangle hold on the promoter, the manufacturer, and the contractor, often forcing them to turn over the lion's share of their profits as compensation for the use of money in the development of their undertakings; and these men in their turn endeavored to even up matters by getting their pound of flesh out of the workman by compelling him to labor long hours for meager compensation. To-day, the laboring man is beginning to come into his own; but if he unwisely takes too great advantage of his growing power, he will "kill the goose that lays the golden eggs", and this will ruin his chance of securing comfort and happiness for himself and his family.

The first step requisite for quieting the existing widely spread popular unrest and returning to normally prosperous conditions is to bring together, so that they may operate in harmony, the financier, the employer, and the laborer; and this must be accomplished primarily by establishing some method of contract-letting and profit-sharing which will be just and equitable to all parties interested. For some time, the writer has been endeavoring to formulate and develop, through communications to the technical press, an ideal method of accomplishing this desideratum; and later herein he will indicate clearly of what it consists. Meanwhile, he will discuss in detail not only the suggestions offered by Mr. Clarke in his timely paper, but also those of various engineers and political economists who have of late been treating the matter in print.

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Various Methods of Contract-Letting.—The most common ways of contract-letting are the following:

A.—Lump sum for complete construction.

B.—Schedule rates for all materials in place.

C.—Actual cost plus a percentage, with some kind of allowance for overhead expenses.

D.—Actual cost of labor and materials plus a percentage, with no allowance for overhead expenses.

E.—Actual cost plus a lump sum, with some kind of allowance for overhead expenses.

F.—Actual cost of labor and materials plus a lump sum, with no allowance for overhead expenses.

G.—Actual cost plus a profit based on small unit prices agreed upon in the contract, as advocated of late by Mr. G. H. Hailey.

H.—Various methods of profit-sharing between the client and the contractor.

Method *A* generally appeals most of all to the client, because it fixes in advance what the construction is going to cost him, unless, perchance, there are important variations in the estimated total quantities of materials, due to lack of sufficient preliminary information or to the encountering of conditions that could not well have been foreseen. It is by no means as satisfactory to the contractor, however, who has to run the risk of being actually out of pocket on the job, in addition to the loss of personal time and effort.

Method *B* is quite an improvement on Method *A*, in that the contractor does not have to guarantee the correctness of the estimated quantities of materials; but the prime objection to Method *A* holds good for Method *B*, as the latter does not prevent the contractor from losing money heavily on the venture.

In the days of hard times, when contractors are willing to take work at low figures, and even below cost, in order to keep their force together, the public, in general, especially as represented by companies and municipalities, is prone to take advantage of them by insisting that work be let by the lump sum, and by throwing on the unfortunate "successful bidder" not only the risk of loss from rising prices of materials and labor and from unforeseen contingencies, but, also, in many cases, from excess of quantities above those given in the specifications. This is accomplished by inserting in the latter a most unjust clause compelling each bidder to verify for himself both the quantities stated and the character of the conditions described. The bidders, hungry for work, accept this clause without comment, but with the mental reservation that, in case of hard luck, they will, by some means or other, obtain extra compensation, even if they have to carry the controversy into the Courts.

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In nineteen cases out of twenty, it is unjust to bidders to ask them to name a lump-sum compensation for doing work, unless provision is made for a variation in the quantities of materials on which they tender. If provision is arranged for such variation, the method of letting is no longer that of the "lump sum", but reduces to a modification of that of "unit prices".

As before indicated, the latter method is certainly the more logical, and yet it is far from being entirely fair to the contractor; because, although it provides against loss through excess above the estimated quantities of materials, it leaves him open to the possibility of still greater loss through changing prices, onerous unanticipated conditions, and disastrous happenings beyond his control. Up to a certain point, the client is the proper party to assume the principal risks inherent to the work, provided that the adverse happenings are really unavoidable by the contractor, and that the latter takes every reasonable precaution against disaster or loss. And yet the contractor should not be altogether relieved from the possibility of loss due to hard luck, because such misfortune is often caused by his dilatoriness, which carries the work into an unfavorable season for field operations. The idea, promulgated of late by certain writers, that it is unjust ever to penalize a contractor, is wrong; because he, like everybody else in this world, should bear the burden resulting from his own carelessness, negligence, or incompetence. The owner surely has some rights—and this is one of them. A liberal limit of total cost which can be increased or reduced properly, in order to provide for an increase or decrease in the estimated total quantities of materials, will prevent the owner from being excessively overcharged, and still will give the contractor every opportunity to come out whole in any case except that of extraordinary hard luck.

Method *C* is wholly objectionable to the client, in that it places him entirely at the mercy of the contractor and his men. The allowance for overhead may be either an assumed percentage, or the actually computed amount in complete detail, the former being generally the less objectionable. Even if the contractor is perfectly honest and has the best will in the world to keep down the cost for the benefit of the client, his employees will not have that desire. In effect, they say to themselves and to each other, "What is the use in my exerting myself unduly? The more the work costs, the more money the 'old man' makes". The writer knows this to be the case, for some years ago he had to let a large contract for foreign work at cost plus a percentage; and although the contractors themselves tried to do the honest thing at all times, their men "loafed" to such an extent that the final cost of the construction was atrociously high; and he had occasionally, on his own responsibility, to discharge some of the contractor's employees,

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including once the field superintendent. This method of letting work involves asking too much of frail human nature.

Method *D* involves less labor in cost-keeping than Method *C*; but, otherwise, it is open to the same general objection.

Method *E* is almost as unsatisfactory to the client as Methods *C* and *D*, except that the contractor's reward for his own iniquity is a fixed quantity and not in direct proportion to the extent of that iniquity.

Method *F* involves a slight improvement on Method *E*, but only to the extent of a little simplification in bookkeeping and perhaps a reduced opportunity for "squeezing" the client.

The writer readily acknowledges that during war times, when the trend of the market for both materials and labor was rapidly upward, no contractor could have afforded to take work either for a lump sum or by unit prices. Unfortunately, in order to bring the war to a successful conclusion, a vast amount of public work had to be done with the utmost despatch, irrespective of what the cost might be; hence, the Government had no choice at all in the matter, and, consequently, it let many millions of dollars' worth of contracts at "cost plus a percentage" or "cost plus a lump sum". If the true history of all such contracts was ever written and made public, the nation would stand aghast at the extravagance they involved; and those two methods of contract-letting would receive the universal condemnation of all intelligent, disinterested persons.

As stated by one of the discussors, when a contractor has simultaneously two or more contracts, one of which is on the "cost plus" basis and the other or others on either the "lump-sum" or the "unit-price" basis, he will naturally put his best and most energetic men on the latter, and will shift the lazy and incompetent ones to the former. This practice has become so well established by custom that the "cost plus" contracts have been dubbed "hospital jobs"; and it appears that the nickname has stuck.

Is it not obvious that any one who lets a contract on the "cost plus" basis places himself absolutely at the mercy of the contractor and the contractor's employees? It is true that the specifications often contain restrictions which tend to lessen the contractor's power to take advantage of the client; but their enforcement would be very troublesome, and would generally involve litigation with its attendant delay and expense.

Most people will acknowledge that the percentage of truly conscientious contractors is not overwhelmingly large, but how much smaller is that of truly conscientious workmen! The writer does not deny that there are workmen who always give a *quid pro quo* and who are upright and honorable in all their dealings; but, alas, they are

sadly in the minority. Their number is so small that they are unable to induce their co-laborers to exert themselves any more than they are compelled to, unless they are paid by the job instead of by the day or hour. Mr. Waddell.

By the way, when it is practicable, such a scheme of paying the workmen is an improvement on that of time compensation, because it provides a great incentive to effort; but, at the same time, it also serves as a strong temptation to scamp the work. With close supervision, however, and a strict enforcement of the clause in the specifications relating to the taking out and replacing of defectively built work, the employees soon learn, through the fines and penalties enforced by the contractor, that scamping does not pay, and that the old adage of honesty being the best policy is just as applicable now as it was when first enunciated.

Method *G* involves only a very slight modification of that of "cost plus a lump sum", the said lump sum being replaced by another sum obtained by adding together the products of the actual quantities of all the materials by certain small unit prices agreed on in the contract. Although the writer concedes that this method is undoubtedly the best of all the straight "cost plus" methods, it possesses all the serious objections inherent thereto.

In addition to those previously indicated, there might be mentioned the fact that any straight "cost plus" basis effectively cuts out competition, and advantages a favored few of the larger and more experienced contractors, rendering it difficult for the smaller and less experienced ones to secure any work, except through some other method of letting. It ought to be evident to any one possessed of ordinary vision that such a method will militate toward cutting the "small fry" contractors out of bidding; for when an owner is willing to let a piece of work on any straight "cost plus" basis, he naturally will want to award it to a large contractor of means, who has an established reputation for fairness and efficiency. That would practically mean letting all contract work without competition, and American contractors, as a body, would object seriously to any such procedure. It is true that the owner might call for competitive bids on the basis of having each bidder name a lump sum as a fixed net fee, and awarding the contract to the competitor who quotes the lowest figure; but the adoption of such a method would often result in serious trouble, delay, and expense.

Under the heading *H*, a number of profit-sharing methods have been tried and have proved to be more or less satisfactory. All of them necessarily presuppose a careful accounting of cost from start to finish. Unless the contract between the two parties clearly indicates how every main detail of cost is to be computed, there will be trouble before the work is finished. For instance, in respect to plant —does the contractor furnish it free of charge, or does he receive

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rental for it, with the rental charged as one of the items of cost of the work? How about paying for repairs and renewals to plant? Who stands the expense of these items—does the contractor, or is it charged as an item of cost of doing the work? Again, if extra work is done under the contract, how is it to be counted when making the final settlement?

It requires the service of an expert consulting engineer or that of an experienced contractor to draft a contract and specifications which will provide, in a manner satisfactory to both parties, for all possible contingencies. With such papers, however, and with a close system of cost-accounting, this general method of profit-sharing is the most satisfactory scheme for contract-letting, which can be evolved.

None of the modified methods of the "cost-plus" system, involving some means or other of profit-sharing, which has yet been tried in practice, can be said to be entirely satisfactory to the owner, though possibly so to the contractor, in that they all fail to put a limit on the total cost of the construction or to penalize a contractor who, through either wilfulness or carelessness, allows the cost of construction to pass the bounds of reason. It does not suffice to stipulate in the contract that, when the total cost passes a certain amount, the allowance for profit is gradually to be reduced until a certain minimum limit, however small it may be, is reached. The setting of that limit leaves the contractor in a position to take life easily and to avoid personal worry after his hard luck has attained to a certain magnitude; for, subsequently to that, he will lose nothing but his time and the possible use of his plant on some remunerative contract, while the owner will have to pay whatever additional amount the job may cost. On this point the writer knows whereof he speaks; because one of the war-time contracts engineered by his firm was, of necessity, let on that basis, and the results thereof are simply sickening. The client was left at the mercy of the contractor, and the total cost proved to be excessive.

From the preceding it is evident that the "cost-plus", the "lump-sum", and the "unit-price" methods of letting contracts are not only faulty, but also unjust to one or other of the two parties to the agreement; consequently, the question arises: "Is there not some method which will be just and fair to both?" That question, the writer claims, can truly be answered in the affirmative; but before proceeding to explain such a method in complete detail there will be presented a statement of the main requirements of an ideal system.

Salient Features of an Ideal System of Contract-Letting and Profit-Sharing.—The essential requirements of an ideal type of contract are as follows:

First.—It must provide a means of sharing with the workmen on an equitable basis the total net profit on the job.

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Second.—It must set some kind of a limit to the total cost of the work, so as to prevent a careless or conscienceless contractor from running up the expense to an outrageously great amount.

Third.—It must reduce to a minimum the chance of the contractor's being out of pocket on the completion of the work, unless such condition is due to his own carelessness or lack of push.

Fourth.—It must retain all the advantages of competitive bidding, so as to give every capable and worthy contractor who is desirous of figuring on the work an even chance of securing the contract.

Fifth.—It must provide an incentive for the contractor and all his assistants and workmen to use every legitimate effort to make the work as inexpensive as possible, without violating in any manner the requirements of the specifications.

Sixth.—It must provide a just and equitable basis of payment for a possible increase in the estimate of total quantities and for adjusting satisfactorily to all concerned the reduction of payment due to a possible diminution thereof.

Seventh.—It must ensure that the owner will be acting for his own best interests by aiding the contractor in every possible way to complete his work quickly and inexpensively, provided, of course, that it is done in such a manner as to guarantee the attainment of the owner's ultimate purpose, as expressed in the specifications.

Eighth.—Its provisions must be such as to keep constantly in good humor every one connected with the construction.

Ninth.—Its method of final settlement of accounts must be clear, simple, and easy of application; and the keeping of them during the progress of the work must be no more complicated or expensive than it would be in the case of any ordinary "cost-plus" contract.

Description of the Ideal Method.—Let the specifications, which should invariably be drafted by an engineer who is acknowledged to be an expert in the class of work covered in the proposed contract, be complete and thorough in every detail, recording all that is known concerning the governing conditions; pointing out all features about which there is any uncertainty; tabulating as accurately as possible the estimated quantities of all the materials that will probably enter the construction; providing a justly drawn clause for unclassified work and the payment therefor; calling for each bidder to submit in full detail his estimate of actual cost of doing the work by applying to all quantities of materials given in the specifications, unit-cost prices

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(termed Schedule A), each price containing a proportionate share of any contingency allowance that may have been made, totaling the products so as to form "Sum A", and adding thereto the amount of profit which he has decided to ask, thus making "Sum B". This last amount, in reality, will represent the bidder's tender; but to it, if it is accepted, there will be added a profit for the owner, exactly equal to that asked for by the successful bidder, and another profit or bonus for the employees, amounting to a previously fixed percentage (say, 20 or 25) of the sum of the aforesaid profits of the contractor and the owner, thus making "Sum C". This last sum is the temporary limit of total expenditure on the part of the owner, based on the assumption that the approximate quantities of materials given in the specifications are correct. The ratio r of "Sum C" to "Sum A" is to be applied to each of the unit prices used in the preparation of the cost estimate, in order to obtain the list of unit prices (termed "Schedule B") to apply temporarily to the actual quantities of materials in the completed construction, when making the final adjustment of accounts.

If there are any items of expense of construction not covered by the list given in the specifications, the clause of the latter relating to "Unclassified Work" will take care of them. That clause should stipulate that, for all such unlisted items, the actual cost of labor and materials therefor, without any allowance for superintendence or overhead, is to be recorded; and to it is to be added later 30% of its amount to allow for superintendence, overhead, and the various profits. This sum is to be added to the total value of the actual quantities of all the materials listed in the specifications figured at the proportionately increased unit prices as given in "Schedule B"; and the result, "Sum D" (with a single modification explained hereinafter), will be the final limiting cost to the owner and the basis for computing the net profits to be divided between the contractor, the owner, and the workmen.

The specifications, of course, will contain a clause providing a surety company bond for the faithful performance of the work and for guaranteeing the client against having to pay more than the limiting sum agreed on (as finally modified).

Method of Profit-Sharing Contract.—The following method of profit-sharing between the contractor, the owner, and the employees is to be adopted:

An accurate estimate of cost of every detail of the work from start to finish is to be kept by the contractor and verified by an accountant in the employ of the client, so that the total profit on the job may be ascertained by deducting this total cost from the maximum figure named in the contractor's tender and afterward embodied in the contract (modified, however, as hereinafter described). This profit, less

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the amount of the employees' bonus, is to be shared between the contractor and the client as indicated in the profit diagram, Fig. 1. It should be clearly understood that every direct and indirect expense to which the contractor is put in doing the work, after the contract is signed, is to be included in the cost—all overhead expenses of every kind, plant deterioration, traveling expenses, supervision, and salaries, excepting only that the contractor himself is not entitled to any salary. In the case of a firm being the contractor, the head of that firm should receive no salary; but if any of the juniors devote their time exclusively to the job, it would be legitimate to allow them reasonable salaries, equivalent to what would have to be paid to regular assistants. All such matters, however, should be stipulated in the contract.

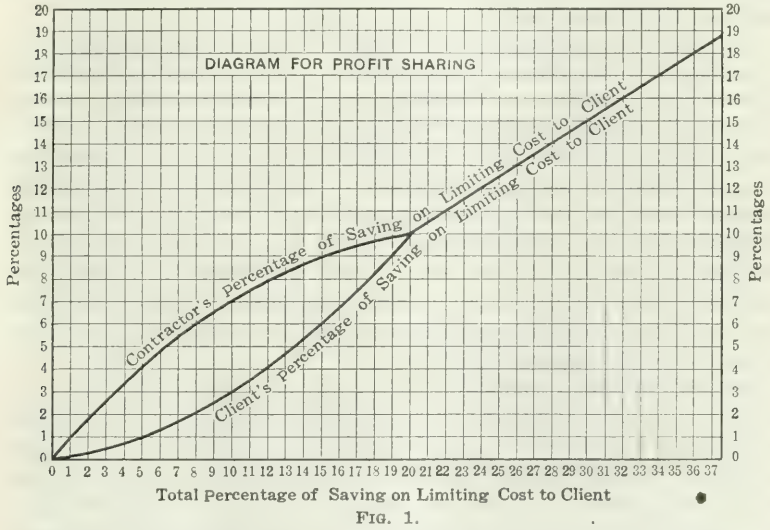
DIAGRAM FOR PROFIT SHARING
Percentages
Percentages
Contractor's Percentage of Saving on Limiting Cost to Client
Client's Percentage of Saving on Limiting Cost to Client
Total Percentage of Saving on Limiting Cost to Client

FIG. 1.

In order to determine, after the entire job is finished, the amount due the contractor, "Sum C" is to be subtracted from "Sum D", and the ratio which this difference (either a positive or a negative quantity) bears to "Sum C" is to be figured and adopted in the use of the diagram of "corrective ratios" (Fig. 2) for the said difference.

Application of Corrective Ratio.—There are two reasons for applying this corrective ratio;

First.—In the case where the actual quantities of materials exceed the estimated ones of the specifications, it would be hardly fair to the owner to apply to the excess those unit prices which produce his tentative limiting expenditure.

Second.—In the case where the actual quantities of materials are less than the estimated ones, it would be unjust to the contractor to

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use the high unit prices on the diminution quantities, not only because of the great difference between these and the unit actual costs, but, also, for the reason that the total overhead charges would be about the same for the diminished amounts as for the estimated total quantities.

In the corrective ratio diagram (Fig. 2), it will be noticed, that, after the ratio of value difference (due to increase or diminution of quantities of materials) reaches 0.2, the "corrective ratio" remains constant at 0.8, which corresponds approximately to actual cost conditions. The object of this is to provide that the contractor shall not be too much benefited by an abnormal increase in quantities, nor, on the other hand, shall he obtain too much advantage because of an abnormal diminution thereof.

To utilize the corrective ratio diagram (Fig. 2), look on the line of abscissas for the ratio of cost difference, pass vertically upward to the curve (or right line, as the case may be,) then horizontally to the extreme left vertical, which will indicate the corrective ratio required. Next, multiply the previously computed difference by this corrective ratio, and add the result to or subtract it from "Sum C". The result, "Sum E", will be the finally corrected limit, from which must be subtracted the total actual cost so as to determine the amount of profit to be divided. The first step in such division is to set aside the employees' share on the basis of percentage agreed on; and the next is to divide the remaining profit between the contractor and the owner, as per the profit diagram (Fig. 1).

The size of the percentage of the declared profit to allow the employees will depend on the character of the work covered in the contract in respect to the proportionate division of the cost between materials and labor. Under ordinary conditions, the division is about half and half, in which case the employees' percentage should be from 20 to 25; but where the labor cost preponderates these figures should be increased, and where the materials cost is the greater they should be diminished.

In respect to the division of this bonus among the employees, the following method is suggested:

Only those workmen or assistants of any class who have stayed on the job, either until its completion or until their services were no longer needed, should participate in the profits; and the amount of the share of each such workman and assistant should be in the proportion which his total earnings on the work bears to the grand total of the earnings of all those employees who so participate. As, in any good business organization, a record is always kept of the amount of salary or wages paid to each employee on any contract, it would require only a few hours of extra work for the bookkeeper, after the job is finished, to compute each man's proportionate share of the bonus.

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There is an additional protection against possible loss which might be given to the contractor under certain conditions, especially on work to be done in a foreign country. If it be anticipated that during construction any large general rise in the price of labor is likely to occur, thus greatly augmenting the total cost of the work, the limiting total expenditure of the owner, as hereinbefore finally adjusted, should be increased by an amount figured thus:

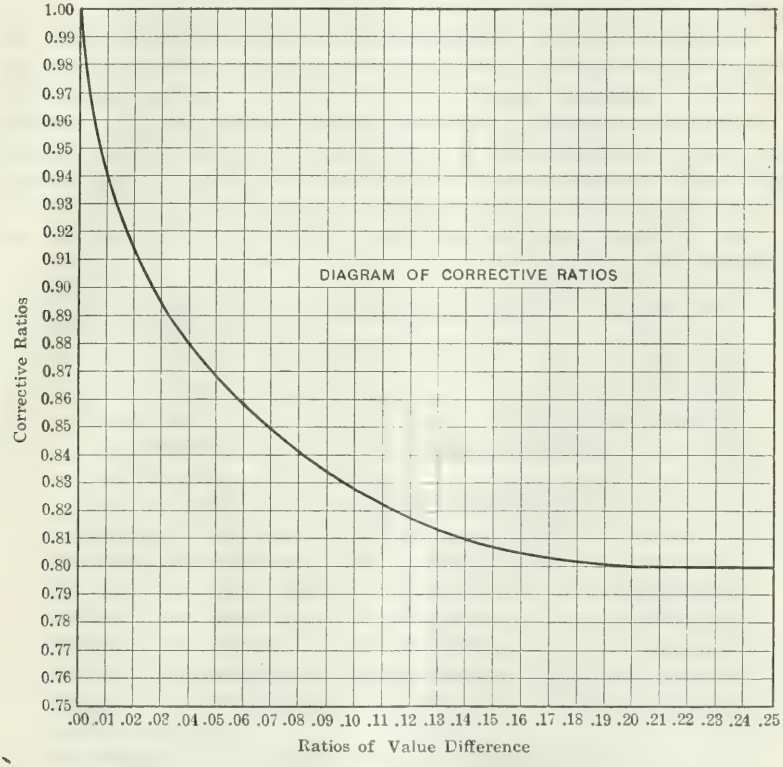

FIG. 2.

Determine the average current wages for common labor at the time of letting the contract, and the average paid therefor during the entire time occupied by the construction, and call the ratio of these two averages r'; then figure the total amount of salaries and wages paid to employees from start to finish and call it W. Then,

$$W\ (r' - 1) \times 1.2$$

will be the amount required, the factor, 1.2, covering a fair allowance for overhead and profits.

Mr. Waddell.

It has been suggested of late in the technical press that the contractor himself should be paid a salary by the owner, in addition to whatever profit he may make on the job. Such a policy would be simply suicidal on the part of the owner, for if the work were to be handled badly, the contractor might continue to earn money while the owner would be losing heavily. The writer has met with just such a case; hence, in this particular, he certainly knows whereof he speaks. Such a practice should never, under any conditions, be countenanced by either the owner or his engineer.

Exemplification.—In order to illustrate the *modus operandi* of this method of profit-sharing, let us assume the following case, in which the estimated quantities are exceeded. For the purpose of simplification in figuring, round numbers have been assumed for both the quantities of materials and the unit costs thereof. The job is one of railroad construction, and the number of items is intentionally limited for the sake of convenience.

The following are the quantities of materials supposed to be stated in the specifications:

Earthwork, measured in cutting.....	1 000 000 cu. yd.
Loose rock, " " "	100 000 " "
Solid rock, " " "	40 000 " "
Concrete in structures.....	10 000 " "
Wooden trestle	2 000 lin. ft.
Structural steelwork, erected.....	500 000 lb.

The tender of the successful bidder was as follows:

	Quantities.	Schedule A.	Totals.
Earthwork	1 000 000 cu. yd.	@ $0.50 =	$500 000
Loose rock.....	100 000 " "	@ $1.00 =	100 000
Solid rock	40 000 " "	@ $1.50 =	60 000
Concrete	10 000 " "	@ $20.00 =	200 000
Wooden trestle..	2 000 lin. ft.	@ $50.00 =	100 000
Steelwork	500 000 lb.	@ $0.08 =	40 000
Total estimated cost ("Sum A") =			$1 000 000
Profit required, 10% =			100 000
Tender ("Sum B") =			$1 100 000
Allowance for owner's profit =			100 000
Employees' profit, 25% of $200 000 =			50 000
Temporary limit ("Sum C") =			$1 250 000

$$\text{Ratio, } r = \frac{\text{Sum } C}{\text{Sum } A} = \frac{\$1\,250\,000}{\$1\,000\,000} = 1.25.$$

Mr. Waddell.

The proportionately increased unit prices, therefore, will be as follows:

Schedule B.

Earthwork	$0.50	× 1.25	=	$0.625
Loose rock	$1.00	× 1.25	=	$1.25
Solid rock	$1.50	× 1.25	=	$1.875
Concrete	$20.00	× 1.25	=	$25.00
Wooden trestle	$50.00	× 1.25	=	$62.50
Steelwork	$0.08	× 1.25	=	$0.10

The actual quantities of the materials in the completed job were, as follows:

Earthwork	980 000 cu. yd.
Loose rock	110 000 " "
Solid rock	50 000 " "
Concrete	10 500 " "
Wooden trestle	2 100 lin. ft.
Steelwork	480 000 lb.

In addition, there was done by the contractor certain "unclassified work" which actually cost him for labor and materials $20 000.

The revised estimate is, therefore, as follows:

Earthwork	980 000 cu. yd.	@	$0.625	= $612 500
Loose rock	110 000 " "	@	$1.25	= 137 500
Solid rock	50 000 " "	@	$1.875	= 93 750
Concrete	10 500 " "	@	$25.00	= 262 500
Wooden trestle	2 100 lin. ft.	@	$62.50	= 131 250
Steelwork	480 000 lb.	@	$0.10	= 48 000
Unclassified work	$20 000 × 1.3			= 26 000
Summation, or "Sum D"				= $1 311 500

The difference between "Sum D" and "Sum C" equals

$$\$1\,311\,500 - \$1\,250\,000 = \$61\,500.$$

$$\text{Ratio of difference} = \frac{\$61\,500}{\$1\,250\,000} = 0.0492;\ \text{say, } 0.05.$$

For this ratio, the diagram, Fig. 2, gives a corrective ratio of 0.865, which multiplied by $61 500 gives $53 198, say, $53 200, making the corrected limit, or "Sum E", = $1 250 000 + $53 200 = $1 303 200.

If the total actual cost of the work, including that of the unclassified work without allowance for superintendence or overhead, amounted to $1 100 000, the total profit would be:

$$\$1\,303\,200 - \$1\,100\,000 = \$203\,200.$$

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Of this, the joint share of the contractor and the owner would be:

$$\$203\,200 \div 1.25 = \$162\,560;$$

and the employees' bonus would be:

$$\$203\,200 - \$162\,560 = \$40\,640,$$

which is exactly 25% of $162 560.

In respect to the division of this last amount between the contractor and the owner, its ratio to total cost is:

$$\$162\,560 \div \$1\,100\,000 = 0.148, \text{ or } 14.8\%.$$

From the diagram for profit division (Fig. 1), we find the division of this percentage to be:

Contractor	8.9%
Owner	5.9%

This makes the total payment by the owner to the contractor:

$$\$1\,100\,000 \times 108.9 = \$1\,197\,900$$

Let us now take a case where there is a diminution in the estimated quantities of materials. Using the same case as before in respect to estimated quantities and tender, we shall assume the following actual quantities of materials:

Earthwork	1 050 000	cu. yd.
Loose rock	50 000	" "
Solid rock	20 000	" "
Concrete	8 000	" "
Wooden trestle	1 800	lin. ft.
Steel work	450 000	lb.

The cost of the unclassified work was $20 000, as in the preceding case.

The revised estimate is, therefore, as follows:

Earthwork	1 050 000 cu. yd.	@	$0.625 =	$656 250
Loose rock	50 000 " "	@	$1.25 =	62 500
Solid rock	20 000 " "	@	$1.875 =	37 500
Concrete	8 000 " "	@	$25.00 =	200 000
Wooden trestle	1 800 lin. ft.	@	$62.50 =	112 500
Steelwork	450 000 lb.	@	$0.10 =	45 000
Unclassified work	$20 000 × 1.3 =			26 000
Summation, or "Sum D"	=			$1 139 750

The difference between "Sum D" and "Sum C" equals

$$\$1\,139\,750 - \$1\,250\,000 = -\$110\,250$$

$$\text{Ratio of difference} = \frac{\$110\,250}{\$1\,250\,000} = 0.0882$$

Mr. Waddell.

For this ratio, the diagram gives a corrective ratio of 0.835, which, multiplied by $110 250, gives $92 059, say, $92 100, making the corrected limit,

$$\text{"Sum E"} = \$1\,250\,000 - \$92\,100 = \$1\,157\,900.$$

If the total actual cost of the construction, including that of the unclassified work without allowance for superintendence or overhead, amounted to $880 000, the total profit would be:

$$\$1\,157\,900 - \$880\,000 = \$277\,900.$$

Of this, the joint share of the contractor and the owner would be:

$$\$277\,900 \div 1.25 = \$222\,320,$$

and the employees' bonus would be:

$$\$277\,900 - \$222\,320 = \$55\,580,$$

which is exactly 25% of $222 320.

In respect to the division of this last amount between the contractor and the owner, its ratio to total cost is:

$$\$222\,320 \div \$880\,000 = 0.253, \text{ or } 25.3\%.$$

From the diagram for profit division, (Fig. 1), we find the division of this percentage to be on the "fifty-fifty" basis, making the total payment by the owner to the contractor $= \$880\,000 \times 1.1265 = \$991\,320$.

Advantages.—The advantages of this method of contract-letting are as follows:

First.—While it is true that the client at the outset does not know exactly what the work is going to cost him, he is positive that it will not cost him materially more than a certain amount, provided his engineer's estimate of quantities is about right, as, generally speaking, it certainly ought to be.

Second.—The client has the satisfaction of feeling that even if, in his opinion, the limit determined by the contractor's bid is excessive, and that the final net profit on the job, in consequence, will be too large, the said net profit will be shared between them on a "fifty-fifty" basis.

Third.—While the client is bound to pay a certain percentage of the joint profit as a bonus to the contract's employees, generally he will not be out of pocket thereby, but, on the contrary, he will gain; because the incentive that the prospective bonus gives to all hands to labor energetically will save in the total cost much more than the amount of the bonus.

Fourth.—All the advantages of competitive bidding are retained by this method, because the fully capable competitor who tenders the

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lowest amount for "Sum B" should be awarded the contract. All bids will be on exactly the same basis, no modification of the stipulated method of tendering being permitted. It is understood, of course, that the contract will not be awarded to any competitor who does not possess the necessary experience, capital, and plant, and who has not an established reputation for doing good and satisfactory work.

Fifth.—The contractor, if he was not too keen in bidding, knows that there is almost no chance whatsoever of his losing money on the job; because before doing so he would have to use up his allowance for contingencies, his own estimated profit, a profit of like amount allowed for the owner, and a substantial sum representing the employees' bonus. If he ever does use up all these safeguards, the chances are many to one that the fault therefor is entirely his own, being due to his negligence, lack of forethought, or deficiency in energy and push; and in that event he certainly would deserve to be penalized.

Sixth.—All the workmen and salaried employees of the contractor will be satisfied with their job because of the excellent opportunity offered for extra compensation; and they will, of their own accord, work diligently, and occasionally even overtime, in order to expedite the construction. Of their own accord, too, they will run off the job any employee who is a chronic shirker, and they will make it their business to keep everybody busy; because the more cheaply the construction is done the greater will be the bonus to divide among the faithful employees who stick by the work to the finish.

Seventh.—The method of profit-sharing given by the diagram for profit division (Fig. 1), is eminently equitable, in that when the net amount is small, nearly all of it goes to the contractor, and, as it augments, a continually increasing proportion of it goes to the owner, up to the point where the total joint profit amounts to 20%, after which the partition is on a "fifty-fifty" basis.

It will be seen that for a total net joint profit of less than 20%, the following divisions will prevail:

With 5% net, 4% goes to the contractor and 1% to the owner
With 10% net, 7% " " " " " 3% " " "
With 15% net, 9% " " " " " 6% " " "

The 20% point was selected for equal division as being the one above which a contract is generally deemed by contractors to be good, slightly below which it is only fair, and much below which it is bad; for it corresponds to a net profit of 10 per cent. That is as small a margin as is generally deemed safe for any bidder to tender upon, and yet it constitutes a satisfactory profit on a finished job. As for limiting the client's share of the profit to one-half—that is rea-

sonable and just, because he would have no moral right to receive more than his partner, the contractor. If the client's share were allowed to increase indefinitely, it is conceivable that, with a very large prospective total profit, the contractor could save money for himself by making the work more expensive. Mr. Waddell.

Any bidder who tenders on the basis of a profit of less than 10% should be looked on askance by the owner and his engineer; and such a competitor should not be awarded the contract, unless he possesses an exceptionally fine reputation for doing good work and for not quitting his job before finishing it. It is true that during very hard times many worthy contractors are willing to work almost for actual cost, in order to keep their workmen employed; and in such cases the good intention should be recognized in making the award. Nevertheless, it nearly always proves to be unsatisfactory to both the owner and his engineer to let a contract for any piece of construction at a figure below its real value.

Eighth.—The contractor will feel during the progress of the construction that the client is a partner on the job, and that, therefore, he and his engineers will not be likely to be unnecessarily severe in their requirements, also that they will permit the adoption of all legitimate expense-saving expedients, and will not demand too many frills on the finishing.

Ninth.—Owing to the justice and equity involved by this method of contract-letting and profit-sharing, all concerned in the execution of the work will labor whole-heartedly and good-naturedly, avoiding petty squabbles and disagreements; and the result will be earnest, honest effort, a satisfactory piece of construction, and the general contentment of both parties to the agreement.

Tenth.—While this method may have at first glance the appearance of being complicated, it is quite simple; and because of the clear manner in which it is explained herein, it is easily utilized in any actual case by following one step at a time the directions given. The nomenclature of "sums" and "schedules" renders the application of the method very easy. Moreover, it must be remembered that it is to be used only once for each contract, and then only after all the work is completed and the accounts are in proper form. Again, the keeping of the accounts is in no way any more complicated than it would be in case any of the "cost plus" methods were used.

Objections That Have Been Raised to This Method.—A few objections, both orally and in print, have been raised to this proposed method of contract-letting and profit-sharing; but they could not have been well considered, for they are not valid. It will be well before closing this discussion to mention them and show wherein they are untenable. They are as follows:

Mr. Waddell.

A.—It has been asserted that there would be special difficulty in keeping the accounts; but there would be required therefor exactly the same work which would be necessitated by any of the "cost plus" methods. They all involve a correct record of the amount of every legitimate item of expense to which the contractor is put; and the bookkeeping in any case would certainly require an account with each employee, so as to show how much money had been paid him from start to finish.

B.—It has been claimed that the method is too complicated to be useful. On the contrary, as explained previously, it is simple; and the peculiar manner of its presentation herein is such as to render its use merely a matter of following step by step certain clearly written instructions. Moreover, as before indicated, this method of settlement of accounts is used only once, *viz.*, after the completion of the job.

C.—It has been stated that uneducated contractors would have difficulty in understanding the method; but the man who made this claim did the average American contractor a grave injustice. The construction contractors in this country are as bright a body of men as one can find anywhere; and certainly they may be trusted to understand anything in reason that affects their interests.

D.—It has been claimed that most contractors have a general inherent objection to sharing profits with the owner; but a little consideration will show that there is no sharing of the requested profit until after the estimated cost of the work (under the assumption of unchanged quantities of materials) has been exceeded. It was to clarify this situation that the writer changed his original idea of having each bidder name a lump sum (corresponding to "Sum C") as a provisional limit of the owner's total expenditure, and a list of unit prices (corresponding to "Schedule B") which, when applied to the estimated quantities of the specifications, would make the sum of the products exactly equal to the said lump sum, and substituted therefor the method herein described, *viz.*, that of having each bidder submit in detail his estimate of cost and his desired profit, so as to base the competition on the sums thereof thus tendered, and arranging the method of determining the limiting cost to the owner by two additions to the successful bidder's total tender. This change is simply a concession to prejudice, and does not modify the method proposed by the writer in his letter published in *Contracting* in its issue of September 15th, 1919. The only fundamental change between that presentation of the matter and this one is the inclusion herein of a bonus for the employees.

E.—It has been claimed that this method would tend to deceive the owner. He would certainly be stupid if he could not see clearly how its tendency is to cut down the amount that he will pay

for the entire work, and that it will set a just limit beyond which, under the worst possible conditions, he cannot be compelled to pay any more. Mr. Waddell.

F.—It has been stated, as a reason for favoring the "cost plus" system, that bonding companies favor it and oppose all other methods of contract-letting. Naturally, they would do so; because, with the "cost plus" method, their obligation reduces almost to zero, the risk being placed solely on the owner. In truth, with that method adopted, there does not appear to be any valid reason for having a bond at all. When the contractor runs no risk of loss whatsoever, why a surety company bond? Possibly, if the surety companies would consider it from this point of view, they would not oppose the writer's suggested method of contract-letting; for, while it reduces very greatly the possibility of loss to the surety company, it does not destroy that organization's function by removing entirely its *raison d'être.*

G.—The claim has been made that this method is not applicable to cases where no estimate of quantities has been made or for which no specifications have been prepared, and when it is necessary to let the work without delay. The correctness of this claim is granted, but its applicability should be confined to war work, in which the element of time is the principal consideration. In civil life, if a projector of an enterprise is in such haste to start his work that he has to omit the preparation of specifications and estimates of quantities before letting the contract, he certainly deserves what is coming to him in case it proves that he has placed himself in the hands of an unscrupulous contractor on the "cost plus" basis.

H.—It has been pointed out that the law will prevent certain public bodies from letting contracts on a "cost plus" basis, and that such a ruling might apply to the writer's proposed method also. If such really is the case, the remedy would evidently be to modify the law so as to permit of its adoption; but would not the fact of its containing a clause which sets a limit to the contractor's total compensation always render the method legal? This, however, is a question for the lawyers to settle.

I.—A prominent writer on economics has lately issued a tirade against gambling, especially as it applies to contractors guaranteeing owners against the cost of work exceeding the amount of the tender. The writer concurs in this theory to the extent that there should be no gambling on the amounts of assumed quantities of materials, or, in certain cases, on there not being any great rise in the average price of labor, or on possible loss through any reasonably great amount of hard luck; but there is certainly nothing unmoral or oppressive in insisting on his gambling to the extent of guaranteeing against his own incompetency, carelessness, or lack of forethought. The writer is of the opinion that a margin against actual money loss to the contractor, consisting of his estimated amount for contingencies, plus

Mr. Waddell.

his requested profit, plus an equal profit for the owner, plus a substantial bonus for the employees, is amply large to guarantee him against all loss through ordinarily unfavorable eventualities; and that, in ninety-nine cases out of a hundred, if such a margin were exceeded, it would be because of the contractor's own fault.

Adoption of Method.—If this proposed method of contract-letting and profit-sharing is received with favor by engineers, architects, contractors, and builders in general, it could easily be adopted as a standard for the country by calling a small convention with a single representative from each of the leading technical and railroad societies, contracting organizations, bankers' associations, and labor guilds, to discuss the advisability of adopting it (or else some modification of it) and to report the decision of the meeting to the said bodies for their approval. If any large group of clients, such as the railroad companies, were to adopt the method as standard and use it, very soon everybody having construction contracts to let would follow their example, thus making it the universal standard of contract-letting for the country—nor would it be long before other American countries would follow our lead, thus greatly simplifying our business relations with the various American Commonwealths.

Addendum.—In all lines of manufacture the employees should share in the net annual profits of the company; but, in figuring the yearly cost of running the establishment and doing the work, there should be included fair salaries for all the working officers, 6% on the actual amount of cash invested in the plant and business (but not on the total capital stock), an annual allowance for a sinking fund to redeem the bonds or other indebtedness of the organization, and taxes of every kind. The net profit estimated in this way should generally be divided on the basis of one-third to the employees and two-thirds to the company, but sometimes, perhaps, on that of one-fourth and three-fourths. The reason why it should not be split on a "fifty-fifty" basis is that the company has to run the risk of standing a loss in bad years, while the employees do not. Only those employees who are still connected with the company at the end of the year, or who have been discharged in good standing, should share in the bonus; and their proportionate amounts should be computed as previously indicated for the case of construction contracts.

Mr. Woolley.

WILLIAM E. WOOLLEY,* ASSOC. M. AM. SOC. C. E. (by letter).—On reading Mr. Clarke's paper, one concludes that he is an advocate of the "cost plus" system. The writer, however, cannot conceive the idea that the system will ever come into general use.

An owner who intends to build naturally desires to know what he will have to pay, and when he may expect the work to be completed.

* London, England.

Mr. Woolley.

One can say, in fact, that the first question asked is, "How much will the job cost?" How is the engineer to answer when the "cost plus" system is to be adopted? Under the older forms of contract, engineers and architects have been able in the past to answer these two questions fairly accurately, but now, owing to the constantly increasing cost of materials and labor, and the unsettled state of the labor market, we have made use of the "cost plus" system, under which many million pounds' worth of work has been carried out on this side of the water during the war.

In the opinion of the writer, we should regard it as a war emergency only. Under the "cost plus" system, or, as it is known here, the "prime cost plus profit" system, it is impossible to avoid decreased effort. There is no inducement to be economical or industrious. The writer holds that fair competition is a good thing. By competition the general level of effort is raised. One has only to think of the field of sports to arrive at this conclusion. The system is likely to have a disastrous effect on labor, there is no scope for tact in the management of men, there is constant interference and inspection on behalf of the owner, and there is no incentive to seize favorable opportunities of buying materials.

At the same time the writer admits that it is hardly to be expected that contractors generally will take risks to the extent they have in the past. While we have the present unsettled conditions, and with charges increasing all the time, there should be appropriate clauses inserted in the contract allowing for variation in the contract price to meet the increased cost of materials and advances in wages. The writer cannot believe that the "cost plus" system will ever be extensively adopted either in Great Britain or America under normal industrial conditions.

Mr. French.

R. D. French,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been much interested in the discussion of this paper on contracts.

One of the prominent construction firms in Montreal has been utilizing the "cost plus percentage" contract with a rebate to the owner on the actual cost of the work. The following illustrations will make its mode of operations clear:

1. *Actual Cost Exceeds Estimated Cost:*

Estimated cost	$100 000
Estimated commission, 20% of estimated cost	20 000
Actual cost	110 000
Rebate, 10% of actual cost	11 000
Commission earned, $20 000 — $11 000	9 000
Total cost to owner, $110 000 — $11 000 + $9 000	108 000

* Montreal, Que., Canada.

Mr. French.

2. *Actual Cost Less Than Estimated Cost:*

Estimated cost	$100 000
Estimated commission, 20% of estimated cost	20 000
Actual cost	92 000
Rebate, 10% of actual cost	9 200
Commission earned, $20 000 — $9 200	10 800
Total cost to owner, $92 000 — $9 200 + $10 800	93 600

The writer is informed that this contract has given satisfaction in the few cases it has been used, and it appears to do away with one of the objections to the straight "cost plus percentage" form, in that the contractor has some incentive to keep the cost of the work down.

Mr. Ledoux.

J. W. Ledoux,* M. Am. Soc. C. E. (by letter).—Mr. Clarke has presented exhaustively the merits of the "cost plus" system of letting contracts. Under ideal conditions, where the contractor and his force are actuated by fairness and honesty, the system is undoubtedly equitable to both the contractor and the owner, but there is enough evidence in the administration of recent contracts, especially in most of those carried out during and continuing after the war, to prove that the system is capable of the greatest injustice to the owner, and of profiteering. Even where the contractor himself may desire to be fair to his principal, the very fact that it is to his advantage to have the work extend over a long period and cost more than the estimate permeates the entire organization from the superintendents to the lowest paid laborers. It takes away all the responsibility and the impetus for despatch and economical construction.

The writer ventures to say that many of the most important projects carried out on this system might have been done for one-third or one-half the cost under one that would have been to the interest of the contractor to finish the work in the shortest time and at the lowest cost, which is the ideal system. No one has been able to devise a form of contract that will realize this ideal. Therefore, any system that is recommended must be defective in some part.

The usual contracting systems are: The "lump sum", the "unit price", the "cost plus a percentage", and the "cost plus a fixed sum". Some projects lend themselves to the lump-sum bid; such, for instance, as the construction of a pumping engine, a water tower or a lighthouse, where there may be no subsequent necessity for deviating from the original plans. The lump-sum bid is not well suited to the construction of a dam or reservoir or any structure where the quantities are likely to differ from those shown in the original plans, in which cases the unit price system is applicable. Both these systems, however, give the owner the advantage and impose on the contractor

* Philadelphia, Pa.

serious risks. If the work is let to the lowest bidder he must often compete against unduly low bids. If he gets the work he takes all risk of unfavorable conditions, while the owner takes only the risk of increased quantities. In the percentage and "cost plus" methods there is a premium on inefficiency and profiteering, and the advantages lie wholly with the contractor, who takes no risk whatever. Mr. Ledoux.

Engineers have tried to overcome these difficulties without imposing injustice on the contractor. Probably one of the best methods is to invite bids on a unit-price basis, with the proviso that the most acceptable bidder is to be willing to take the work as an alternate on a fixed profit basis. Then the owner, through his engineer, may call in the acceptable bidder and decide on the amount of profit he is willing to pay, regardless of the cost; the work is done on a cost basis plus the fixed profit, and the contractor is permitted to participate in the saving in cost to the extent of from 25 to 50 per cent. This bonus will be sufficient to induce the contractor to expedite the work as much as possible and it will be to the interest of every one connected with the construction to complete the work at the lowest price and in the shortest time. This method may appear to be difficult of application, but the difficulties are more apparent than real. A complete understanding must be had with the contractor as to whether, and how, overhead charges and plant rental are to be included in cost.

Another method of overcoming the difficulty is to select a contractor who has established a reputation for economical construction on the "cost plus" basis, but this method is manifestly unfair to many deserving contractors whose reputations are not so well known.

In any case, the interests of the owner in regard to quality of work can be safeguarded only by honest and capable engineering supervision.

Ernest Wilder Clarke,* M. Am. Soc. C. E. (by letter).—As stated at the time of presenting the paper the writer was principally anxious to induce discussion on a subject which seemed to him to be worth it, especially at this time, and he hoped that such discussion might bring out statements that would indicate the wisdom or otherwise of a more general use of the "cost plus" contract. Unfortunately, with one exception, there has been little but expressions of personal opinion which, while interesting and instructive, are not proof either for or against this form of contract, and he does not consider it worth while to continue to great length further discussion based on such grounds. Mr. Clarke.

Conditions during the war were so abnormal that excessive costs of work should not be charged to any form of contract. Organizations for supervision were improvised over night and naturally did not function smoothly at once. The contractors' organizations were also greatly expanded to care for the vast increase in their work, and

* Baltimore, Md.

Mr. Clarke.

inevitably the two organizations failed, in many cases, to evolve the cheapest methods of doing the work. As the war went on, this handicap was largely overcome, and there were many cases where the supervising engineers and their forces and the contractors and their forces worked together most successfully to devise means to reduce the cost of the work. The high cost of labor and material largely masked such efforts, however, and very little credit will ever be given for real economies effected in spite of adverse conditions. In other words, wartime costs were so far above those of peace times that the fact that they might have been still higher except for the efforts made by the two parties is not appreciated. The contractors are blamed for things of which they really were the victims and over which they had no control. Mr. Frankland states that "this form of contract would 'smell to high heaven'" on some "cost plus" contracts. If this statement is true, it implies connivance on the part of the supervising forces or at least such disregard of their obvious duty that it amounts to connivance, and any one having proof of such conditions should bring the facts to the attention of officers who could punish the guilty persons.

This is not the place to enter into a discussion of the causes of the high costs of war construction, but as the writer had intimate knowledge of conditions on a dozen or more large and small "cost plus" jobs, with large and small contractors at work during 1917 and 1918, he feels himself competent to assert that after the organizations reached a good working condition both parties to the contracts did everything possible to make the work as inexpensive as conditions of the market permitted, and that intentional loading of the pay-roll, loafing on the job, or unnecessary purchases, were neither permitted nor attempted, the contractors' foremen being generally as zealous in crowding their men as on lump-sum and with-price jobs.

The writer does not agree that the use of "cost plus" contracts is an admission of inability to write clear and definite specifications. He had the honor to assist in writing some of the specifications of the New York Board of Water Supply, and is familiar with the extreme care used in order to make those specifications clear and fair to both the city and the contractor; almost every word was scrutinized for an ambiguous meaning, and where any doubt existed as to the exact meaning explanatory phrases were introduced or other more definite word or words substituted. Later, he was in charge of construction under these specifications and learned how many different interpretations contractors' superintendents and engineers could propose for words which he supposed had only one meaning. Most of these disputes were settled amicably, but they were all potential suits, and the mere fact that they were possible is indicative of the extreme difficulty

of writing specifications that will prevent misunderstandings. This is not intended as an argument for the elimination of specifications, but merely as indicating that disputes are not always due to carelessly drawn specifications. Mr. Clarke

The writer still believes that an engineer is, and properly is, "biased" in favor of his employer, the owner; the engineer is hired for the specific purpose of protecting the owner against the contractor's errors, whether or not such errors are intentional, and it is his duty to require the contractor to prove the justice of any claim involving the reading of the contract or specifications; the fact that in many cases the contractor is able to satisfy the engineer that his claims are just and that in such cases the engineer has fought to secure the acknowledgment by the owner of the claims merely accentuates the fact of the prior bias of the engineer.

The one discussion which gave the result of experience with lump-sum and "cost plus" contracts is the best evidence that the latter form, when handled by intelligent and honest contractors, is favorable for both sides to the contract. Presumably, the owners of the jobs covered by Mr. Perry's discussion had engineers or architects representing them and, doubtless, some of these men made suggestions or even ordered methods that resulted in decreased costs.

Engineers are apt to assume a rather sanctimonious attitude toward contractors, to hold that engineers are honest, but that contractors require strict watching to prevent very serious wrong-doing. As a matter of fact, intelligent self-interest forces a reputable contractor to do work which will bear inspection and which will serve as an advertisement of good work and not bad. As men, they are, nowadays, of the same class and education as engineers, and have as high a sense of honor. The writer has had the pleasure of knowing many contractors whose simple word is a guarantee of faithful work and, with such men, the engineer's part on the job is largely that of staking out the work, interpreting the plans and specifications when any ambiguity is discovered, and, in general, making inspections to see that mistakes are not made.

All engineers are expected, and practically all do, work against their own financial interests. Almost invariably they are supposed to push the job in every way they can, to work overtime without any extra pay in order to avoid delay in the work; to order their work so that the construction is not held up—in short to do everything possible to get themselves out of a job and have a chance to hunt for another on their own time.

The writer has yet to find a gang which, as a whole, failed to live up to the expectation that it would in this way work against its immediate and most apparent interest, and he believes that another

Mr. Clarke.

gang which is different only that it is employed by a contractor would perform in the same way, if its chief made it plain that he expected such an attitude, and many contractors have and would make it very plain that that was their expectation. A reputation for such an attitude would be the best guarantee the contractor and his forces could have that, good times or bad, owners would offer them work and that, unlike the engineer, neither the contractor nor his men would have to hunt for work on their own time.

As for the engineer, the "cost plus" offers him a chance to show both executive and constructive ability—to make it evident to the owner that he has saved money for him by methods whereby costs have been reduced, by taking advantage of favorable developments of the site, or by overcoming unfavorable developments at minimum expense.

As chief inspectors, engineers now get credit at the most for negative results—they prevent bad work. It is utterly impossible to evaluate such service in dollars, and little effort is made to do so; their services as designers are largely forgotten after the plans are accepted and work begun and the owner regards them as he regards insurance, as an overhead expense to be cut to the minimum possible. The contractor often thinks that engineers are more or less impractical theorists with whom he has to put up, but who can cause him much loss by insisting on contract or specification interpretations which, to his mind, are unreasonable.

As a matter of fact, on jobs of any size, the engineer is generally the man best informed as to local conditions, methods of work likely to produce good results at minimum expense, and plant required. He has spent weeks, months, possibly years, studying the plans and the site and has a fund of knowledge that is only partly reflected in the plans and specifications. After a lump-sum or item contract is let, the contractor, if he is wise, consults with the engineer and revises his own plans and methods as far as he is convinced of the soundness of the engineer's ideas. Any saving made from such revision generally goes into the contractor's pocket. The owner may gain indirectly by increased speed, but the knowledge of his employee principally benefits the contractor. On a "cost plus" contract, where the owner and the contractor are working in harmony, the engineer becomes a vital part of the machinery. His knowledge and his experience are direct assets to the owner, the contractor, and himself; he ceases to be a spectator of the construction of the work he designs, and is an active participant in methods, plant layout, and responsibility. The writer believes that in endeavoring to relieve owners of responsibility for the contractor's acts, engineers have also relieved themselves so entirely that

they have become mere layers out of work to a large degree, and, as such, receive the wages of subordinates instead of the rewards of creators. Mr. Clarke

It goes without saying that no one form of contract would apply to all work, and this appeared so self-evident that the point was not mentioned in the paper. The generic term "cost plus" was also used because it was recognized that many possible forms of this type existed and many more could be written, although the paper was not an advocate of any particular form. A form such as Mr. Waddell suggests might be applicable to some contracts, other forms to other contracts.

Many schemes for "letting" whereby competition would be secured and the elimination of new firms avoided could be evolved. The engineer's estimate of probable cost would be expected to show the owner in advance the approximate amount of money involved and with the engineer as the owner's representative in full control of expenditures, it would be his duty to keep the expenditures within that limit or explain the reason for failure to do so. It would also be to the engineer's credit if savings were made, and a reputation for ability to keep ultimate costs below the estimate would enable him to demand adequate payment for such service.

The statement has been made repeatedly that under a "cost plus" contract the contractor has no incentive to save money for the owner. This statement does not do credit to the contractor's business sense.

Firms looking for work under such contracts must be able to refer a prospective client to the owners of jobs they have done and be able to show by the cost records of such jobs that economy was effected, otherwise their first job would be their last. As a matter of fact, a record of low costs becomes the most valuable asset of such a contractor and of far more importance than the money that might be made on one or two jobs by running the cost into high figures. The owner is supposed to exercise care in the selection of a contractor and to have the work properly supervised. If he does neither of these things, he may pay a very high price for his structure, but such an owner would be just as likely to be cheated on a lump-sum or item contract.

The use of the "cost plus" form on public work is evidently a much more dangerous thing than on private work, but only on account of the political complication. If experience proves that the form is economical for private owners, some scheme could be devised by which the public also could be benefited.

Some contractors may "enjoy the gamble", but probably they enjoy it the most when they are sure they will not lose; where they have added enough to their item charges or their lump sum to provide for anything short of a general cataclysm and have been awarded the job not on account of their optimism but because of the greater pessimism of the other bidders.

Mr. Clarke.

In the long run, nobody is benefited by a business failure. Some owner may get a job done below cost, but workmen and supply men suffer, and wherever possible the loss is passed on to some other job. Any scheme that tends to prevent failures is a step in the right direction, provided new dangers are not thereby introduced, and "cost plus" contracts can be written and applied so that these new dangers are avoided to a great extent.

If the paper and discussion has the effect of bringing the matter more generally to the attention of engineers, the writer's object is attained. He believes that the "cost plus" contract in one form or another is suitable for use on many jobs and that its use would result in good for owner, contractor, and engineer.

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Paper No. 1442

COMPREHENSIVE PLOTTING OF WATER TURBINE CHARACTERISTICS

By Karl R. Kennison, Assoc. M. Am. Soc. C. E.

Synopsis.

This paper presents a chart for the comprehensive plotting of water turbine characteristics. The use of this chart would enable the manufacturer of a water-wheel to show simply and clearly the performance of his turbine under any and all conditions. It would enable the purchaser intelligently to weigh the merits of different turbines and their compliance with specified requirements.

The arrangement of the paper is as follows:

1. The Need of Comprehensive Plotting.
2. Specific Characteristics of Water Turbines.
3. Description of the Chart.
4. Typical Water Turbines of American Manufacturers.
5. Method of Using the Chart.

1. The Need of Comprehensive Plotting.

The test performance of a water turbine is most commonly represented by a series of curves, one for each gate-opening used in the test. Two, or sometimes three, sets of such curves are usually presented. For all these curves the horizontal scale is laid off in terms of the speed under a given head. For one set the vertical scale is efficiency, for the other, it is power output, and sometimes a third set is added, showing the discharge.

Such curves do not give a sufficiently clear indication of the results that would be obtained at any other gate-opening not included in the tests. An engineer, if he has made a study of the performance of water turbines—but not unless he has made such a study—can interpret these curves and fit them to any desired conditions of head, speed, discharge, etc. Most engineers, however, are busy men, and after learning that the specific speed of a water turbine is that speed at which it would run under a 1-ft. head, if they could imagine it reduced in size until it developed only 1 h.p., they give it up as a bad job.

The following diagrammatic method of showing the performance of a water turbine has proven so simple and convenient to the writer, that he presents it for the consideration of hydraulic engineers and turbine manufacturers.

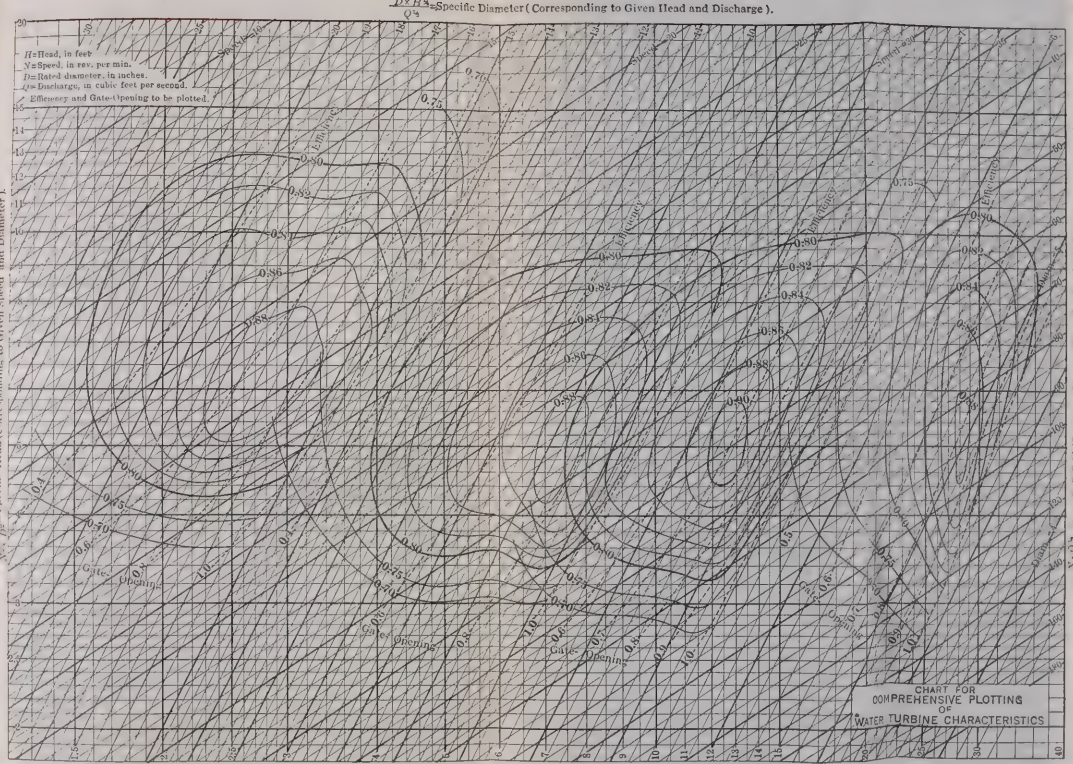
2. Specific Characteristics of Water Turbines.

We are concerned with seven variable quantities. It is, of course, the large number of variables which so complicates the problem. For convenience, let us refer to them by the following letters:

D = rated diameter, in inches.
H = head, in feet.
N = speed, in revolutions per minute.
P = power output, in horse-power.
Q = discharge, in cubic feet per second.
Gate-opening.
Efficiency.

For various reasons large units may show a slightly increased efficiency over small models under all conditions of operation, but it is aside from the purpose of this paper to discuss the extent of such an increase, and no further account will be taken of it. No attempt will be made to discuss the design of turbines, but certain characteristics will be stated as concisely as possible.

It is characteristic of water turbines that when any given design or type is varied in diameter, all the dimensions, of course, being kept in proportion, the efficiency will remain the same, at the same gate-opening, if N is varied directly as $\sqrt{H}$ and inversely as D; and the resulting P will vary directly as $H^{\frac{3}{2}}$ and as D^2, and Q will vary directly as $\sqrt{H}$ and as D^2.


$\frac{D \times H^{\frac{1}{4}}}{Q^{\frac{1}{2}}}$=Specific Diameter (Corresponding to Given Head and Discharge).
H=Head, in feet
N=Speed, in rev. per min.
D=Rated diameter, in inches.
Q=Discharge, in cubic feet per second.
Efficiency and Gate-Opening to be plotted.
Efficiency
Gate-Opening
$\frac{H \times 10^5}{N^2 \times D^2}$=Specific Head (Corresponding to Given Speed and Diameter).
$\frac{N \times D^{\frac{5}{4}}}{H^{\frac{1}{4}} \times 10^3}$=Specific R.P.M. (Corresponding to Given Head and Discharge).
(Numerically Equal to "Specific Speed" at 85% Eff.).
CHART FOR
COMPREHENSIVE PLOTTING
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WATER TURBINE CHARACTERISTICS
$\frac{Q \times 10^5}{N \times D^3}$=Specific Discharge (Corresponding to Given Speed and Diameter).

Hence, the following are twelve simultaneous constants:

Efficiency,	Gate-Opening,	$\frac{N\,D}{\sqrt{H}}$,
$\frac{P}{D^2\,H^{\frac{3}{2}}}$,	$\frac{Q}{D^2\sqrt{H}}$,	$\frac{P}{D^5\,N^3}$,
$\frac{Q}{N\,D^3}$,	$\frac{P\,D^4}{Q^3}$,	$\frac{N\sqrt{Q}}{H^{\frac{3}{4}}}$,
$\frac{P}{Q\,H}$,	$\frac{P^3}{N^4\,Q^5}$,	$\frac{N\sqrt{P}}{H^{\frac{5}{4}}}$,

Of these constants, the last is the one commonly known as the "specific speed." It is that constant which expresses the relation between speed, head, and power.

Of the seven variables referred to, only six are independent variables, since power output and discharge are inter-related through the efficiency. Simplification demands that either P or Q be eliminated from consideration. The writer is aware that he is departing from present practice in eliminating power output and retaining discharge capacity.

Performance curves are usually drawn to show the horse-power output, and with good reason, as the turbine user is looking for results, and the power output is certainly the result in view. However, the problem before the engineer usually involves a given head and a given discharge—to find the turbine which will give the greatest efficiency. The head and discharge are the fundamental quantities with which we have to deal.

For example, the so-called specific speed of a turbine, already referred to, is the criterion by which we judge whether it is of the right type to do its best at suitable speed under relatively low head and high discharge, or *vice versa*. Two turbines may be of the same type in this respect, and yet one may be much more efficient. Hence, the writer claims that the specific speed should properly show the relation between speed, head, and discharge, instead of between speed, head, and power.

Therefore, the type characteristics of a water turbine may be obtained from the previous list of simultaneous constants by omitting those involving the power output. The form in which they are ex-

pressed subsequently has been chosen for convenience of mathematical expression. On account of taking advantage of a fortunate mathematical coincidence, what the writer proposes to call "specific r. p. m.", or the relation between speed, head, and discharge, is numerically equal to the "specific speed" currently expressed as the relation between speed, head, and power, at the efficiency attained by wheels of good design, namely, 88 per cent. Hence, the two terms are practically interchangeable in any general reference to approximate values.

The characteristics are as follows:

$$\text{Specific head} = \frac{H \times 10^7}{N^2 D^2}$$

$$\text{Specific discharge} = \frac{Q \times 10^6}{N D^3}$$

$$\text{Specific diameter} = \frac{D H^{\frac{1}{4}}}{Q^{\frac{1}{2}}}$$

$$\text{Specific r. p. m.} = \frac{N Q^{\frac{1}{2}}}{H^{\frac{3}{4}} \times 10^{\frac{1}{2}}}$$

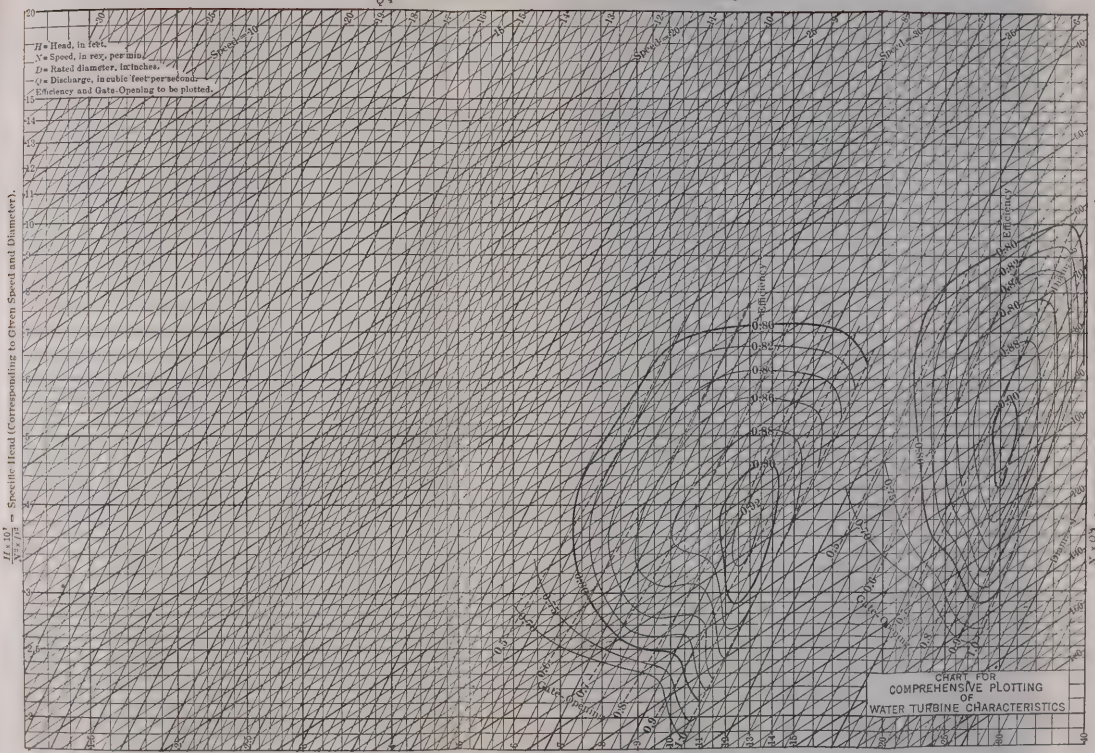
3. Description of the Chart.

The chart is simply a means of plotting efficiency and gate-opening in terms of the four fundamental type characteristics listed previously.

The plotting of test data of any turbine on this chart tells the complete story of the performance of this type or design of turbine, regardless of actual head, discharge, diameter, or speed of the turbine in question; and the plotted result can then be used to show the expected performance under any other desired head, discharge, diameter, or speed.

As the co-ordinate scales for plotting are fixed and can easily cover the entire range of turbine design, the difference between one type of turbine and another is shown at once by the relative location on the chart. For example, nearness to the lower right-hand portion of the chart means high speed and small diameter. Moreover, of several types which plot equally near the lower right-hand portion, the lowest is relatively the fastest and largest and the farthest to the right relatively the slowest and smallest.

$\frac{D \times H^{\frac{1}{2}}}{Q^{\frac{1}{2}}}$ = Specific Diameter (Corresponding to Given Head and Discharge).


H = Head, in feet.
N = Speed, in rev. per min.
D = Rated diameter, in inches.
Q = Discharge, in cubic feet per second.
Efficiency and Gate-Opening to be plotted.
Efficiencies
CHART FOR
COMPREHENSIVE PLOTTING
OF
WATER TURBINE CHARACTERISTICS

$\frac{H \times 10^4}{N \times D^2}$ = Specific Head (Corresponding to Given Speed and Diameter).

$\frac{N \times Q^{\frac{1}{2}}}{H^{\frac{3}{4}} \times 10^2}$ = Specific R.P.M. (Corresponding to Given Head and Discharge)
(Numerically Equal to "Specific Speed" at 88% Eff.).

$\frac{Q \times 10^6}{N \times D^3}$ = Specific Discharge (Corresponding to Given Speed and Diameter).

The vertical and horizontal scales represent the two most important variables with which we have to deal, namely, head and discharge, respectively, so that when the characteristic curves are plotted for any type of turbine, the available range of head and discharge stands out very clearly.

On account of the logarithmic scale of the co-ordinates, the range of operation of any two types may be compared regardless of their location on the chart. For example, the vertical distance enclosed by the 80% efficiency line is the measure of the percentage variation in head in which this efficiency is equalled or exceeded, and, similarly, the horizontal distance is a measure of the percentage variation in discharge. In this respect, a low-speed turbine at the left of the diagram is directly comparable with a high-speed turbine at its right.

4. Typical Water Turbines of American Manufacturers.

Only inward and downward flow turbines are represented. Plate XII shows the characteristic curves of four turbines made by one American manufacturer, covering a wide range of conditions. The specific speeds, at most efficient operation, are, as shown by the chart, approximately 25, 47, 61, and 90. A glance shows that high specific speed is evidently obtained at a sacrifice in efficient range of discharge capacity.

Plate XIII shows the characteristic curves of two turbines made by another American manufacturer, the specific speeds being approximately 77 and 96. The first turbine is highly efficient. It plots low down on the chart, indicating a relatively large diameter. It should be borne in mind that the rated diameter is a more or less arbitrary dimension and that it depends on the amount the runner vanes are cut away. However, the point of measurement of diameter being once determined, the chart gives as exact relations in the case of the diameter as of any other variable. The second turbine is remarkable for its combination of high specific speed and comparatively large range of efficient operation.

A great deal of information as to the performance of American water turbines, of the inward and downward flow type, is condensed in Plates XII and XIII.

5. Method of Using the Chart.

Plate XIV shows at the right a third American manufacturer's turbine with a specific speed of about 91. The method of using the chart is described very briefly, as follows:

Suppose this type of turbine is proposed for the development of an available normal discharge of 240 cu. ft. per sec. per wheel, under a normal head of 30 ft., subject to variation from, say, 24 to 32 ft., often enough to require high efficiency at these heads.

Knowing that $H = 30$ ft. and $Q = 240$ cu. ft. per sec., the scale of "specific r. p. m." can be easily converted into one of speed expressed in revolutions per minute. This scale has been added to the chart at N. Only those speeds are marked, which will give the required 60 cycles per sec. Similarly, the scale of specific diameters can be easily converted into one of diameter expressed in inches. This scale has been added to the chart at D. Only those diameters are marked, which the manufacturer can furnish from stock patterns.

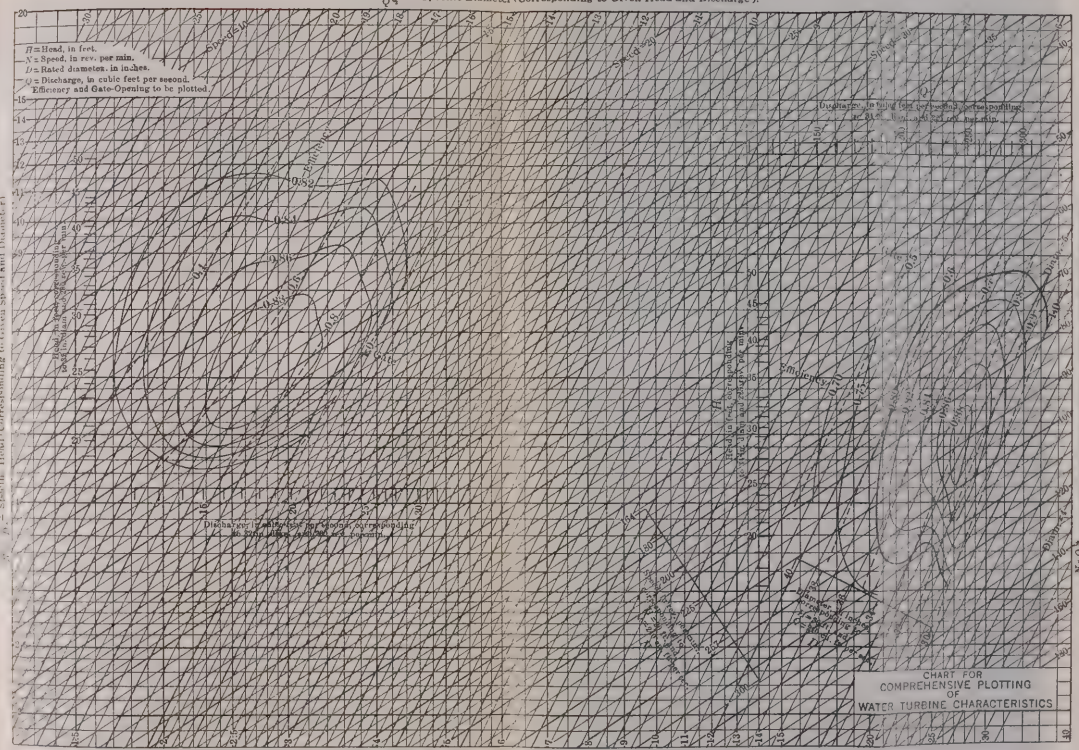
The speed and diameter may then be chosen so that the turbine will do its best under the given conditions of operation. The chart shows that, at 225 rev. per min., a 34-in. wheel will operate at near its peak efficiency and, at the same time, allow efficient operation at a considerable reduction in head and increase in discharge. Hence, this speed and diameter are determined on.

Then, knowing $N = 225$ rev. per min. and $D = 34$ in., the scale of specific heads can be easily converted into one of head expressed in feet, and the scale of specific discharges into one of discharge expressed in cubic feet per second. These scales have been added to the chart at H and Q and enable the performance of the selected wheel to be predicted under all conditions of operation. For example, the chart shows:

Under a 24-ft. head: a gate-opening of about 0.64 will pass a discharge of about 180 cu. ft. per sec., without the efficiency going below 80%; a 0.90 gate will pass about 240 cu. ft. per sec., at an efficiency of 87%; and full gate will pass about 260 cu. ft. per sec., at an efficiency of 84 per cent.

Under a 32-ft. head: a gate-opening of about 0.60 will pass about 190 cu. ft. per sec., without the efficiency going below 80%; an 0.80 gate will pass about 250 cu. ft. per sec., at an efficiency of 88%; and full gate will pass about 290 cu. ft. per sec., at an efficiency of 83 per cent.

$\frac{D \times H^{\frac{1}{4}}}{Q^{\frac{1}{2}}}$ = Specific Diameter (Corresponding to Given Head and Discharge).


H = Head, in feet.
N = Speed, in rev. per min.
D = Rated diameter, in inches.
Q = Discharge, in cubic feet per second.
Efficiency and Gate-Opening to be plotted.
CHART FOR
COMPREHENSIVE PLOTTING
OF
WATER TURBINE CHARACTERISTICS

$\frac{H \times 10^3}{N^2 \times D^2}$ = Specific Head (Corresponding to Given Speed and Diameter).

$\frac{N \times Q^{\frac{1}{2}}}{H^{\frac{3}{4}} \times 10^3}$ = Specific R.P.M. (Corresponding to Given Head and Discharge).
(Numerically Equal to "Specific Speed" at 88 Eff.).

$\frac{Q \times 10^3}{N \times D^3}$ = Specific Discharge (Corresponding to Given Speed and Diameter).

The peak efficiency of approximately 88.5% will be attained at about 0.82 gate, with a discharge of 240 cu. ft. per sec., under a 28-ft. head, and nearly if not quite as high efficiency under the normal head and discharge of 30 ft. and 240 cu. ft. per sec., at 0.79 gate.

To supplement the added scales of head and discharge, a set of curves showing actual horse-power delivered under all conditions could be added very easily, running in the upper left to lower right direction.

To illustrate the fact that distance, regardless of location, on the chart is a measure of range of operation, scales of discharge, in cubic feet per second, and head, in feet, have been added in the left-hand portion of Plate XIV to show the performance of a 32-in. wheel of low specific speed, operating at 200 rev. per min. Note that the spacing of these discharge and head scales is identical with that of the discharge and head scales for the high specific speed wheel at the right.

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Paper No. 1443

THE SILT PROBLEM OF THE ZUNI RESERVOIR

By H. F. Robinson, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Elwood Mead, Clarence S. Jarvis, George M. Post, and H. F. Robinson.

Synopsis.

This paper discusses the silting up of reservoirs in the arid part of the United States, with special reference to the Zuni Reservoir in New Mexico, which shows a loss of capacity of 60.7% in 12½ years.

In all reservoirs built in the western part of America, and especially in those built in a stream bed by damming the channel, the silt problem is one that must be faced sooner or later; and unless some means can be found for removing the silt deposits, the accumulation will impair, materially, the storage capacity of the reservoir, and, at some date, controlled by local conditions, the entire basin will be filled to the brim.

This is particularly true of the ephemeral, flashy streams, dry a considerable portion of the time and subject to freshets and floods at intervals. At such times the floods of water coming down the dry beds will erode the bottoms and banks to a great extent, much more so than if the stream bed was continually wet, and the accumulation in the reservoir will be much greater when the reservoir is in the stream channel, as the entire débris of the erosion, whether carried in suspension by the water or rolled along the bottom, will be deposited in the reservoir.

The Zuni Reservoir is in the Zuni Indian Reservation in Western New Mexico, about 45 miles south of Gallup. It is formed by the

construction of a dam across the Zuni River, a tributary of the Little Colorado.

The dam is a combination rock and hydraulic fill, and was built by John B. Harper, M. Am. Soc. C. E., Superintendent of Irrigation in the U. S. Indian Irrigation Service. The rock fill has a height of 80 ft., or from the bottom of the puddle trench to the top of the parapet, 110 ft., the length over all being 780 ft. There are 40 000 cu. yd. of rock in the fill, and 60 000 cu. yd. of the hydraulic fill. At the point where the dam is built, the stream has cut through a lava flow, and the two abutments are composed of the lava or basalt.

The reservoir was built between 1902 and 1907, the gap in the stream being closed in 1906, from which time water was impounded; but, until August, 1909, only a small portion of the reservoir capacity had been stored.

In September, 1909, the water in the reservoir reached the spillway lip, and, it is supposed, found vertical cracks in the basalt south abutment of the dam, allowing the water to reach a stratum of sand under the basalt, causing a subsidence of the mesa, which wrecked the spillway and damaged the end of the dam.*

As originally constructed, the spillway of the Zuni Reservoir was at Elevation 1 000, but when the new spillway was built, it was placed 9 ft. lower. It was planned that when more storage capacity was needed some form of stop was to be placed in the spillway, either a permanent weir, or gates of some kind, with the top edge at the old elevation, and proper foundations were placed under the floor of the spillway to allow for this.

The quantity of silt brought into this reservoir is very great, and in the 12½ years since the channel was closed, has amounted to about 4.86% per annum of the reservoir capacity to the elevation of the present spillway.

Soundings were made in the reservoir in June, 1910, January, 1912, January, 1914, and January, 1918, in order to ascertain the quantity of silt deposited. With the exception of the first period (from June, 1906, to June, 1910), when the run-off was not carefully measured, the run-off records have been kept with considerable accuracy, and

* An article describing this failure appeared in *Engineering News*, Vol. 62, pp. 597 to 600, and an article relative to the flood of that time, and an attempt to find the relation between the records of the run-off and rainfall appeared in *Water Supply Paper No. 269* of the U. S. Geological Survey, p. 206. This same article also appeared in *Engineering News*, Vol. 64, pp. 203 and 204.

as the last three sets of soundings were taken through the ice, and the measurement points located with a transit, it is thought that the results obtained are of considerable accuracy.

During the first period, which was 4 years, the quantity of silt deposited was 1 804 acre-ft., an average of 451 acre-ft. per year. During this period, the estimated flow was 56 140 acre-ft., which is thought to be slightly in excess of the actual flow; but, assuming it to be correct, the quantity of silt deposited was 3.2% of the run-off.

During the second period (1½ years), the silt deposit amounted to 1 340 acre-ft., or 894 acre-ft. per year. The run-off was 24 630 acre-ft., and the silt was 5.4% of this flow.

The third period (2 years) had a run-off of 14 360 acre-ft. The silt deposited was 1 056 acre-ft., or 528 acre-ft. per year, and the percentage of silt to the run-off was 7.3, the greatest in any period.

During the fourth period, the conditions of run-off were very different. The seasons were wet, and although the run-off was many times the quantity of the former periods, the silt deposited seems to have been considerably less than in former years. There is found to be a total deposit for the 4 years of 1 307 acre-ft., or 329 acre-ft. per year, but as the run-off for the period was 226 195 acre-ft., the silt percentage was only 0.58.

For the total period of 12½ years, the run-off has been 326 648 acre-ft., the silt deposit, 5 507 acre-ft., and the average silt content, or deposit of the water, 1.88 per cent.

In addition to this silt deposit, which only represented the quantity left in the reservoir, a considerable quantity has been carried out of the reservoir through the gate-valves, and, during the last period, it is probable that a large quantity carried in suspension passed over the waste-way. Considerable water has been wasted through the valves back into the river, such waste having been necessary, especially of later years, in order to keep the valves clear, and some muddy water was drawn off for use on the fields.

It was estimated by the Engineer in Charge that each year during the third period, at least 50 acre-ft. of silt went to the fields and at least three times this quantity went back to the river, through the valves, with the waste water. How much has been carried out during the past four years, or what proportion of silt was in the water passing

over the waste-way, there are no records to show, but the quantity is not inconsiderable. (See Table 1.)

TABLE 1.—Silt Deposit in the Zuni Reservoir from Original Closing of Channel in 1906, to January 1st, 1919.

All Capacities and Silt Deposits in Acre-Feet.

	Original, 1906.	June, 1910.	Jan., 1912.	Jan., 1914.	Jan., 1918.	Jan., 1919.	Total period.
Total silt deposit	0	1 804	3 144	4 200	5 507	6 186	6 186
Length of period, in years	.....	4	1½	2	4	1	12½
Silt deposit for period	.....	1 804	1 340	1 056	1 307	678	6 185
Silt per year, average	.....	451	894	528	327	678	494
Total run-off	.....	56 140	24 630	14 360	226 195	5 323	326 648
Percentage of silt in run-off	.....	3.2	5.4	7.3	0.58	12.5	1.88
At Contour 990:							
Capacity	10 230	8 409	7 069	6 013	4 706	4 028	4 028
Percentage remaining of original capacity	.....	82.3	69.2	58.8	46.0	39.3	39.3
Lost capacity per year in percentage of original	.....	4.4	8.8	5.2	3.2	6.7	4.86
At Contour 1 000:							
Capacity	15 811	13 969	12 632	11 562	10 218	9 540	9 540
Percentage remaining of original capacity	.....	88.4	80.0	73.1	64.6	60.3	60.3
Lost capacity per year in percentage of original	.....	2.9	5.6	3.48	2.1	4.3	3.17

Life of reservoir, Contour 990 = 21 years. Spillway at Contour 991. Capacity between Contours 990 and 991 = 507 acre-ft.
Life of reservoir, Contour 1 000 = 32.4 years.

Considering the reservoir at its present construction (see Table 2 and Fig. 1), the original capacity at Contour 990 (1 ft. below the present spillway lip), was 10 213 acre-ft. At the end of the first four years, it was 8 409 acre-ft., a loss of 4.4% each year, leaving the capacity 82.3% of the original. At the end of the next year and a half the capacity was 7 069 acre-ft., a loss of 8.8% per annum, with

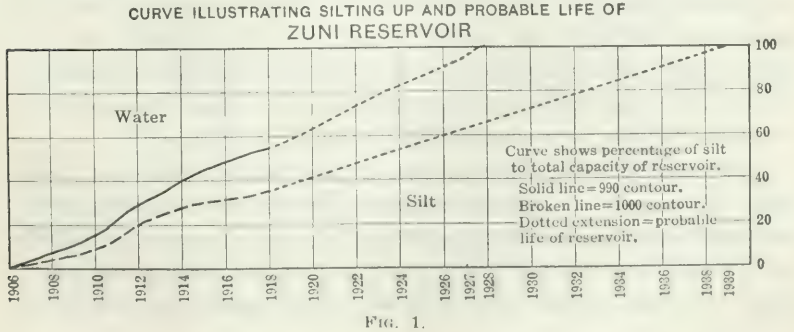

Fig. 1.

69.2% of the capacity of the reservoir remaining. At the end of the third period (two years later), the capacity was 6 013 acre-ft., a loss of 5.2% per annum. At the end of the next period, the capacity was 4 706 acre-ft., or only 46% of what it had been 11½ years before; and, at the end of the last period, the capacity was only 4 028 acre-ft., 39.3% of its original capacity. The average capacity lost each year was 4.86%, and the average lost per year for the entire 12½ years was 4.86 per cent.

This means that the life of the reservoir to Contour 990 will be a little more than 21 years, of which 12½ years have passed. At this rate, the reservoir will be silted to within 1 ft. of the spillway level by June, 1926, and within a couple of years from the time of this writing (1919), the storage capacity will have been so impaired that it will not store enough water to carry the project over a normal year, and certainly not enough from a wet year to tide the project over a dry one.

TABLE 2.—Capacities, in Acre-Feet, of the Zuni Reservoir at Various Times, Showing Its Reduction by Silt Deposits.

Contour.	1906.	June, 1910.	January, 1912.	January, 1914.	January, 1918.	January, 1919.
940	0	.....	.....	.....	.....	.....
945	7	.....	.....	.....	.....	.....
950	53	.....	.....	.....	.....	.....
955	198	.....	.....	.....	.....	.....
960	591	0	0	.....	.....	.....
965	1 345	185	1	.....	.....	.....
970	2 569	990	249	16	0	.....
975	3 758	2 256	1 190	407	53	12
980	5 563	3 920	2 673	1 720	786	706
985	7 763	5 997	4 679	3 625	2 330	1 652
990	10 213	8 409	7 069	6 013	4 706	4 028
991	.....	.....	.....	.....	5 213	.....
995	12 887	11 062	9 721	8 664	7 336	6 660
1 000	15 811	13 969	12 632	11 562	10 218	9 540

Similar data have been worked out with the spillway restored to the 1 000-ft. contour. (See Table 2.) At the present time, there remains only 60.3% of the original capacity of the reservoir, and its extreme life is about 32½ years, or, by 1939, the entire reservoir will be filled to the brim—unless, in the meantime, some method can be devised to remove the silt, or a considerable portion of it.

The quantity of silt deposit shown by these measurements may seem to be very large to one who has not made a study of the streams of the arid Southwest, especially the ephemeral streams where there is little or no clear water run-off, the entire annual discharge being quick floods after the summer rains or in the spring when the snows of the

higher elevations melt. Lake McMillan, in New Mexico, is reported to have silted up 60% in 15 years.* Taking the Zuni Reservoir at the 990-ft. contour, the percentage of filling would be about 70 in 15 years, and using the capacity of the reservoir at the 1 000-ft. contour, about 48 in the same time. In the reports concerning the Austin Dam, it is stated that in the 4 years, from 1893 to 1897, the reservoir was silted up 38% of the original capacity, and during the period of 7 years, from 1893 to 1900, it had silted up 48 per cent.

Mr. W. W. Follett† gives 1.66 as the mean percentage of silt carried by the Rio Grande, at San Marcial, N. Mex., from 1897 to 1912, inclusive. For the same period, the maximum was 4.14% in 1911 and the minimum 0.78% in 1905. He mentions the maximum for one day—October 7th, 1911—as 14.87% and the monthly mean of 7.9% for the month of October, 1911. It is probable that the maximum mentioned by Mr. Follett, namely, 14.87%, would represent about the maximum quantity ordinarily carried in suspension. There are exceptions, as will be shown later.

Some experiments made by Rollin Ritter, Assoc. M. Am. Soc. C. E., Engineer in Charge of the Zuni Reservoir, leads him to the following conclusions regarding the silt-ladened waters of the Zuni River, and some general conclusions regarding the quantity of silt that may be transported in some of these Southwestern streams. A report made by him in 1914, to the writer, contains the following statements:

"The silt-ladened waters being heavier than the clear, ofttimes appeared at the tunnel outlet when the reservoir surface was clear and a small or moderate flood was entering the reservoir. During one period, when the water wasted appeared very heavily charged with silt, a sample was taken at the tunnel outlet. This bottle test showed percentages of silt by volume at various times about as follows:

Time after collecting sample.	Apparent % of silt.
1 day	55
7 days	35
1 month	25
3 months	17½
4 months	16

"At the latter time, the silt in the bottle was compact and even showed lamination marks and probably no more shrinkage would have

* Third Annual Report of the U. S. Reclamation Service.
† *Engineering News*, January 1st, 1914 (Vol. 71, p. 18).

occurred. In the earlier periods when a sliver was plunged through the mouth of the bottle into the silt, the line of demarcation between the surface of the silt and supernatent clear water was not sensible to the touch. Unfortunately, the bottle containing the sample froze and burst, and the observations for a longer period could not be made.

"The above may illustrate why so many engineers regard the bottle test as worthless. It may also explain why some enormous percentages have been reported at times from various sources, *i. e.*, the silt as first observed will eventually shrink to one-third or even one-fourth the volume observed in the freshly gathered sample.

"No tests are available to solve the question of how much was carried by the water in suspension, and how much rolled down the stream bed. The latter is, however, a large proportion of the former."

Some measurements made by the writer on Bonito Creek near Fort Defiance, Arizona, gave something over 13% of sediment after the sample had stood 30 days. The most remarkable instance of large quantities of sediment carried by water was observed in the Moencopi Wash on the Painted Desert of Arizona. During a flood on this stream, a sample was dipped up from about the center of the thread of the stream and allowed to settle for two days, and remarkable as it may seem, at the end of that period, there was 15% of turbid water and 85% of sediment, showing that the flow of the stream was practically liquid mud. The sample was upset before it had time to settle further, but it is probable that a settlement of 30 days would have shown not less than 50% of solid matter.

Unfortunately, the Zuni Reservoir is situated in the bed of the stream, and all silt whether carried in suspension or rolled along the bottom of the stream bed is caught in the reservoir, except the comparatively small quantity that passes out through the valves, and the lighter portion of the sediment which is held in suspension and passes over the spillway at times when the water is flowing into the full reservoir. We have here a fairly large basin and a moderately high dam, and the entire closing up of the channel. "The greater the height of the dam or size of reservoir, the greater will be the rate of filling."* This is the opinion of the late H. F. Labelle, M. Am. Soc. C. E., after an examination of the silt question in many reservoirs and behind many dams both in the United States and in India. It

* *Engineering News*, Vol. 60, p. 212.

will be seen, therefore, that in the case of this reservoir all conditions are favorable for the retention of large quantities of silt.

The area of the water-shed tributary to the Zuni Reservoir has been quoted as 650 sq. miles. From this, however, should be deducted the area above the Ramah Dam and a considerable area of rolling country where the drainage flows into natural shallow tanks or basins. Unless there is a cloudburst, there is no overflow from these natural basins, and the precipitation rarely if ever runs into the Zuni River. It is thought that 500 sq. miles more closely represents the real area of the water-shed draining into the Zuni Reservoir.

There are no records of the rainfall in this drainage area, and a study of the run-off divided into periods, as given in Table 3, will show that the precipitation is very irregular. During the two years, January, 1912, to January, 1914, the average run-off was less than 7 200 acre-ft., while from January, 1914, to January, 1918, the average run-off exceeded 56 000 acre-ft. It is probable that the average annual rainfall of the water-shed is between 10 and 16 in., and until rainfall records are available on the water-shed, no relation between the rainfall and the run-off can be determined.

It is generally believed that the quantity of silt carried by any stream is largely in proportion to the magnitude of the floods, but, according to the records at Zuni, little relation is found between the magnitude of the flow and the silt carried. During the period from 1906 to 1910, with an average annual run-off of more than 14 000 acre-ft., the percentage of silt deposit was 3.2. In the next year and a half, with an average run-off of 16 420 acre-ft., the silt deposit was 5.4 per cent. During the next period of two years, with a minimum run-off amounting to a little more than 7 000 acre-ft. per year, the silt deposit was 7.3% of the total, while in the last period of four years, with an average annual run-off of more than 56 000 acre-ft., it was a little more than one-half of 1% (0.58%), and for the total period of 11½ years, the solids deposited from the run-off were 1.71 per cent.

In estimating the probable life of the reservoir, the percentage of silt in the run-off has been neglected, and the annual average quantity of silt actually deposited has been used.

In an article* by Herman Stabler, M. Am. Soc. C. E., and Mr. Horatio M. Parker, regarding the silt carried by the Kaw River, they

* *Engineering News*, Vol. 63, p. 644.

state that, in June, 1908, the river carried 41.4%, and in May, June, July, and August, 1908, 80.7% of the total for two years. From this they deduce: "The extraordinary influence of great floods, in erosion and denudation, is clearly shown." While this may be true of the flow of large rivers, it does not seem to be true in the case of the flashy streams of the Southwest, where the lesser volume of water in a year, coming in a few periods of run-off, seems to carry the greatest quantity of silt.

TABLE 3.—Showing Run-off, in Acre-Feet, of the Zuni River, at Blackrock, N. Mex.

Total run-off for 1906 and 1907, 38 000 acre-ft.
January to June, 1908, 2 200 acre-ft.
July to September, 1908, 2 900 acre-ft.
October, 1907, to June, 1908, 500 acre-ft.

	1909.	1910.	1911.	1912.	1913.	1914.	1915.	1916.	1917.	1918.
January	.....	2 540	1 460	.....	.....	1 750	48	11 771	40	40
February	.....	.....	1 340	350	.....	2 930	4 604	23 560	712	880
March	.....	500	5 200	.....	1 380	5 190	26 695	71 880	1 232	1 240
April	.....	.....	.....	400	1 420	800	13 737	13 177	354	0
May	.....	.....	.....	.....	.....	1 200	5 078	55	134	28
June	.....	.....	.....	.....	.....	200	808	.....	98	558
July	1 160	1 800	8 820	1 800	1 380	7 200	1 204	1 444	.....	.....
August	5 500	2 100	1 660	1 190	300	800	1 859	1 275	600	860
September	2 840	150	760	.....	3 480	500	740	1 320	505	760
October	.....	.....	1 340	380	2 200	450	.....	22 068	102	422
November	.....	.....	.....	.....	120	.....	28	38	.....	515
December	.....	.....	.....	.....	50	130	10	.....	.....	20
	9 500	7 090	20 580	4 030	10 330	21 150	54 811	146 588	3 777	5 323

Total run-off, 1906 to 1918, inclusive, 326 779 acre-ft.

It has occurred to the writer that the great difference in the quantity of silt carried by the water may be explained, to a certain extent, by the fact that during the periods of lesser run-off the ground in the water-shed and the stream bed are very dry and, consequently, easily eroded by the floods, and that during the periods of greater run-off, more evenly distributed, the ground and the stream channels are kept moist and, therefore, are not subject to as great erosion during the beginning of heavy precipitation and run-off.

A remedy is not easily suggested. The situation involves a serious fact, not a theory, and at a very early date it will be necessary to put into practice some method of getting rid of the silt. Otherwise, a

project built at a cost of more than $500 000, under which more than 3 500 acres are now being cultivated, and products valued at more than $68 000 were raised last year by the Indians, will have to be abandoned inside of 20 years. It will be necessary to raise the spillway elevation within a year or two, for the entire capacity of the reservoir to-day is only slightly in excess of one year's need for the present area of cultivated land.

Certain remedial measures have been suggested. One of these is to construct a set of small overflow dams in the stream above the reservoir. Even if these could be successfully constructed, and maintained at a reasonable cost, the silt accumulation of one year would fill a small one, and the ultimate filling of the main basin would be postponed only a short time.

It also has been suggested that much of the silt comes from the rapid cutting of the banks of the stream, and that rip-rapping these banks at the critical points would prevent further erosion and reduce the quantity of silt coming into the reservoir. The main stream is too long to make this feasible, and it has not been observed that the waters were any less heavily charged with silt at the head-waters of the feeder streams than in the main stream.

It also has been suggested that large sluices be installed at a low elevation, and the silt be gotten rid of in that manner; but observation in cases where dams have given way and the entire content of water run out through the opening, discloses that the quantity of silt sluiced out has been almost negligible.

The St. Johns Reservoir, in Arizona, the Austin Reservoir, and others, would indicate that a channel no wider than the opening, and extending back for a short distance, with almost vertical sides, would be the result.

The only plan that sounds at all feasible is that outlined roughly by the Consulting Engineer, J. H. Quinton, M. Am. Soc. C. E., when he was on the project some years ago.

This plan was to have a floating pipe line, connected with the lowest valve on the tower, now below the bottom of the reservoir. On a barge out in the lake, a cutter and nozzle were to be mounted, with a gasoline engine for power to operate the cutter and a small vacuum pump. With a flow started through this pipe line, no power would be required to continue such flow other than the operation of the vacuum pump

which would be placed at the highest point on the pipe line. What is really a suction dredge would thus be provided, and if all the water drawn from the reservoir was taken out in this manner, large quantities of silt could be removed.

Possible objections would be that in dry seasons there would not be sufficient water to waste, and all the water, heavily ladened with silt, would have to run into the canal, and, without a silt trap, it might fill up the two siphons in the canal. If it did not do this, much silt would be deposited in the canal and cause considerable expense and trouble for its removal.

It is thought that some means of combatting the silt will be evolved, but, with the rapid filling up of the reservoir, steps should be taken at once to find a solution of the problem.

DISCUSSION

Mr. Mead.

ELWOOD MEAD,* M. AM. SOC. C. E. (by letter).—This paper deals with the most serious problem of water conservation in the arid Southwest. The author shows that the Zuni Reservoir which cost $500 000, has lost six-tenths of its capacity in 12½ years, that the reservoir near Carlsbad, N. Mex., has lost more than half its capacity in 15 years, and that, in a few years more, both these basins will be mud-flats of no value for irrigation.

The same menace confronts nearly every reservoir built in the channel of streams in the southern half of the arid region. The Rio Grande at times carries so much silt that it runs in waves, like the tailings away from a stamp-mill. In its channel, and in those of other sediment-laden streams, great reservoirs have been built, and millions of dollars have been or are being spent on irrigation channels, clearing, leveling, and draining lands, building houses, and planting orchards. The benefits to come from all this endeavor and expense depend on the water supply being permanent, on the storages being maintained to practically their full capacity. Every one knows that, unless some plan of desilting is devised and put into operation soon, this will not be the result, and that the life of all these storages will be comparatively brief.

These great undertakings are examples of the happy-go-lucky practice of acting first and thinking afterward; of refusing to see things that are disagreeable. The people of the Southwest wanted irrigation; they wanted the deserts made green, but they were not willing to wait until some feasible plan of desilting was worked out. The water was needed; the immediate results would be good, let the future and the people who would buy these farms, take care of themselves.

The conservation of water for irrigation and power purposes stands on an entirely different footing in the northern and southern halves of the arid region. In the northern half, reservoirs can be placed in the channels of running streams and endure. The silting process is so slow that the cost of removing the sediment, if it ever became necessary, would not be prohibitive. This is shown by the fact that the natural lakes in the streams of this region have been there for unnumbered centuries without being filled or greatly reduced in area by silting. Yellowstone Lake in the channel of Yellowstone River, Jackson Lake in the channel of Snake River, Fremont and Wind River Lakes in the tributaries of Green River, Lake Tahoe in the channel of the Truckee River, and Klamath Lake in the channel of the Klamath River, are illustrations of conditions that make reservoirs a safe basis for both agriculture and power development in practically all the region north of Arizona and New Mexico.

* Berkeley, Cal.

Mr. Mead.

South of this region, there are no natural lakes in the channels of streams. The heat, the aridity, and the torrential character of the rainfall, are friends of erosion, and help to explain the high percentage of silt which these streams carry at certain seasons. To build storages in these channels, in the absence of some definite plan for insuring their permanence, ought not to continue. It is one of the matters which ought to be investigated, and one in which actual conditions in all existing reservoirs ought to be known and published, in order that this menace may be faced and, if possible, some plan worked out of averting the danger. Mr. Robinson has rendered a service to the whole country by publishing the facts.

Referring to some plan for desilting, the writer's attention has been called recently by L. C. Hill, M. Am. Soc. C. E., former Engineer of the U. S. Reclamation Service, who had much to do with the preparation and execution of the plans for the Elephant Butte Reservoir in the Rio Grande, to the fact that provision has been made for lengthening the life and usefulness of this reservoir in another way.

This provision is as follows: Instead of building a reservoir capable only of meeting the demands of the land it was to irrigate, the height of the dam was increased so as to give the reservoir a large excess capacity. This excess is so great that it is estimated it will be 60 years before the silt deposited will begin to reduce the storage capacity below the needs of the irrigated area. When that time arrives, the foundations and side-walls of the reservoir are of such a character that the height of the dam can be raised in successive stages so as to prolong the useful service of the reservoir for 300 years.

Such duration removes this reservoir from the list of temporary storages. Although it does not render it permanent, it does make it feasible financially to meet the expense of later storages higher up stream. The cost of the impounding dam of the Elephant Butte Reservoir would not be lost when the reservoir is silted up, as the dam will have great commercial value for the generation of power.

The writer has also learned that provision for a similar excess storage was made at the Roosevelt Dam and Reservoir. For many years silting will not encroach on the required volume of stored water and, as at Elephant Butte, it is possible to increase the thickness and height of the dam so as to prolong the useful life of this reservoir for centuries.

The writer did not have the Roosevelt Dam in mind in his reference to this subject, as it is in the higher reaches of a stream where erosion can be greatly retarded by preservation of the forests and vegetable growth on the water-shed of the reservoir. He was thinking

more of reservoirs built on the lower courses of streams, but makes this correction in order to avoid having his comments given a broader implication than was intended. Mr. Mead.

CLARENCE S. JARVIS,* ASSOC. M. AM. SOC. C. E.† (by letter).—Due to the deterring influence that will be exerted on meritorious irrigation projects now under way or about to be undertaken, as well as to the discouraging effect on the stockholders of existing systems pending a satisfactory solution for each enterprise where the silt problem is a serious menace, the subject dealt with in this paper has increased in immediate importance with the publishing of such conclusive data as are furnished by the author. The unfortunate condition that obtains concerning the relative aridity and the silt-bearing power of the streams, the greatest erosion usually taking place during the torrential run-off from the barren, parched slopes at the time of those occasional fitful downpours or "cloudbursts" indigenous to the Southwestern States, producing temporary rivers in the desert for a few hours, serves to increase the difficulty; for where it is most important to conserve the entire flood run-off and to maintain the maximum depth of storage to reduce evaporation, the introduction of vast quantities of sediment not only reduces the available storage capacity, but also increases the relative loss from evaporation, due to the increasing shallowness. Mr. Jarvis.

Factors that will enter into the solution or discussion of the problem, such as the relative quantities of water to be stored compared with the quantities passing the dam site, the gradient of the drainage and irrigation channels, the character of the sediment, the accessibility of the reservoir site for the delivery of construction materials and equipment, the limiting heights for the future storage, and the productivity of the land served with water under the various irrigation systems, will vary so widely as to require an independent solution for each case.

Apparent and Actual Capacities.—It is well established that the actual yield of a reservoir may be greatly in excess of the apparent capacity, due to the lateral extension of the stored water into the loose talus slopes so common to mountain valleys, or in some cases to the lateral infiltration into great depths of permeable soil. Inspection of the recently uncovered slopes of a receding reservoir will disclose numerous seeps and springs yielding the water from the underground reservoir, which represents the difference between the apparent and actual capacities. The writer has observed that, in the case of numerous glacial moraines on the borders of mountain lakes being converted into reservoirs, the excess storage capacity thus obtained in basins ranging from a few acres to more than a hundred acres amounted to from 5 to 20% of the apparent capacity. Also, the apparent capacity of a river valley filled with rock detritus and sediment is practically nothing; yet the wonderful success of submerged dams for the purpose of bringing

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† Now M. Am. Soc. C. E.

Mr. Jarvis.

the underflow from such river deposits to the surface, proves that the safest and best reservoir sites, unaffected by the usual evaporation losses, are in the voids of sedimentary deposits. Perennial springs issuing from nearly horizontally stratified rock, or even from talus slopes on mountain sides, are occasionally successfully retained during the non-irrigation period, with a resultant increased flow when released.

While the percentages of voids in sedimentary deposits may vary from 40% for clean gravel and sand to 20% for fine silt, and in extreme cases the range may be outside of these limits, the actual yield for stream flow may amount to nearly 40% in the case of coarse material, and to practically nothing in the case of fine silt. In the latter case, the main losses of moisture are by surface evaporation, which is rendered continuous through capillary attraction until all of the excess or free moisture is removed from the sediment; while the seepage is so slight as to be lost by evaporation.

It is evident that a reservoir completely filled with clean coarse sand and gravel might retain from 30 to 40% of the original volume for effective storage, while the same basin filled with fine silt, or with coarse sediment mixed with enough fine silt to produce the maximum density, would retain possibly 15% of voids, but might yield less than 5% of the original volume of storage.

Removal of Sediment.—The ordinary methods of excavation with standard equipment, at several reservoir sites where the writer has been engaged as engineer, have resulted in an average cost varying from $1.00 to $0.10 per cubic yard of earth in large deposits. Using the lower unit price, the cost of removing 1 acre-ft. of sediment, approximately 1 600 cu. yd., would be $160, which could not be justified under current irrigation practice. The original cost per acre-foot of storage capacity in successful irrigation projects may run as high as $50, or as low as $2; with the rise in values of property in irrigation districts and with the reduction of maintenance expense on the canal systems, larger annual appropriations may be diverted each year for the recovery and maintenance of storage capacity, but a low unit cost for handling the material is imperative.

Apparently, the only method that may be used successfully for this purpose is some adaptation of hydraulic sluicing. The cost of pumping 1 acre-ft. of water against 100-ft. head, with power delivered at 1 cent per kw-hr., and an efficiency over-all of only 50% is slightly less than $2 for power. If the water carries a burden of fine sediment amounting to 15% by weight or 10% by volume, then the power charge for such pumping will amount to approximately $20 per acre-foot of dry sediment. It is not unusual for the plant and operation charge to equal the power charge; therefore, under such conditions, the cost per acre-foot of sediment removed would be $40. Fortunately, the sediment is seldom required to be elevated as much as 100 ft.; it is

usually sluiced out by way of the lowest channel, and the percentage of sediment carried need not be limited to 10% by volume if appropriate grades and equipment are used. Mr. Jarvis.

Consider now the type of reservoir ordinarily provided in the channel of a stream. For this case, assume an elongated site with a steep gradient in the channel, a high dam, and a stream run-off two or three times the original volume of the storage. Proper manipulation of the outlet gates, which should be of ample dimensions, together with judicious use of hydraulic pumps and jets from hydraulic giants mounted on a barge for use around the edges of the reservoir during the irrigation season and especially while the surface of the water is receding, reducing the larger deposits of silt and maintaining a turbid outflow, could be adjusted so as to keep pace with and correct the silting process. The alluvial silt thus distributed upon the irrigated land would be of immense value as a fertilizer; the canal system would have less percolation losses, and there would be less likelihood of the canal breaking than where clear water is used.

Under some conditions the plan suggested by Mr. W. W. Follett for combating the sedimentation menace at the Elephant Butte Reservoir might be advisable. This consisted of a conduit constructed in the bed of the reservoir and connecting the outlet with the Narrows about midway along the axis of the basin. This conduit would convey the sediment from the up-stream portion of the site to the outlet without allowing it to be deposited within the down-stream portion at any time. The up-stream area therefore would be an effective silt pocket for the main storage basin, and insure the delivery of turbid water for irrigation purposes. With such a conduit constructed through a reservoir site and provided with standpipe connections at intervals, the practical plan suggested by J. H. Quinton, M. Am. Soc. C. E., as Consulting Engineer for the Zuni Reservoir project, as related at the conclusion of the paper, the desilting of a reservoir site would be economically and effectively accomplished under ordinary conditions.

The type of reservoir that has no waste outflow lacks the advantage of the channel sluicing; but in other essentials the problem is similar to that previously described.

The preferable way to prevent excessive sedimentation is to reduce erosion; but this method certainly has its limitations. There is no doubt that the habits of some streams could be modified in this particular by planting suitable trees, willows, shrubs, and other vegetation, and protecting the banks against lateral erosion by barriers and deflecting structures wisely placed; and the value of the timber thus grown in the unforested regions would go a long way toward compensation for such improvements. The natural processes of erosion, however, will continue to some extent; and wherever reservoirs occupy the stream channel the burden of sediment must either be diverted, car-

Mr. Jarvis.

ried through by some natural or artificial sluicing, or else the reduction of storage capacity must be included in the reckoning.

Conclusions.—The writer cannot concur in the view expressed by Elwood Mead, M. Am. Soc. C. E., in his discussion, to the effect that the desert places should have remained barren until some feasible plan of desilting was worked out. If the future of the irrigation projects now most seriously beset with trouble from sediment should necessarily include abandonment, then the views so clearly expressed by Mr. Mead would meet with universal approval. Taking into account the influence of reservoirs upon the habits of a river, cutting off the peaks from floods and distributing some percentage of the flow that would otherwise be wasted to lands from which the underflow, when definitely established as the result of irrigation, will continue steadily for months or even years, one must conclude that the service from such reservoirs in establishing underground reservoirs of increasing capacity, compensates in part for the loss of capacity due to sedimentation. So important is the seepage flow as the result of irrigation that it is not uncommon for the owners of such projects to specify in the contracts of sale to homeseekers that the resulting seepage water may be collected by the distributing company and used as an integral part of the water supply for the project. The writer has observed that, where water was used copiously upon bench or plateau lands, the visible return flow became considerable after four or five years; and lands at lower elevation within the same project were being supplied in part from this source.

Reservoirs of elongated form along the main channel, through which large floods may flow freely for considerable periods, and which can be filled with the remaining flow after the stream has clarified in large measure, can be regarded as permanent and of practically constant capacity, irrespective of the quantity or character of the silt, as has been demonstrated in the Assuan Reservoir on the Nile. Inasmuch, however, as the regular and dependable habits of the Nile place this reservoir in a class by itself, the miniature reservoir projects that resemble the Assuan in essential features will require a somewhat modified treatment, and with varying degrees of success.

The main river channel through an elongated reservoir site represents a greater percentage of the original capacity than is realized at first consideration; and there can hardly be a well-founded claim that this portion of the reservoir will become a level mud flat. Some lateral erosion will occur naturally if the reservoir gates, of ample size, may be kept open for considerable periods, and this result would be obtained by the direct sluicing of the natural stream.

Departure from the simple process of direct sluicing through a reservoir basin immediately involves additional equipment or structures, such as conduits, pipe lines, pumping plants, barges, etc.; and

Mr. Jarvis

it would be necessary to make thorough study and skillful designs to avoid the uncertainties of experimentation. Reduced, however, to the simple requirement that the average silt content of the outflow shall be equal to the average burden of the inflow, the problem is capable of satisfactory solution in all except the extreme cases. The popular notion of the sediment carried by a stream is often based upon the observation of the maximum burden. The Rio Grande with an average silt content of 1.66%, and the Gila with an average of 1.3% during years of observation, have to receive and carry on the occasional contributions of liquid mud from their tributaries; and without the published data obtained by reliable observers, it is certain that a much higher silt content would be credited to these rivers by those who had casually observed the heavy floods.

The silt problem along the rivers of the Southwest is not more serious than the drainage problems that arise in irrigated districts; yet settlers and land owners and engineers cheerfully accept the challenge of these two lurking enemies of the irrigation project. Too often the first cost of the system so far overruns the estimate that there are no funds available, during the first years of operation, to give the needed attention to these matters; but it is fully realized that the accumulation of sediment in the one case, or the accumulation of seepage water in the lower lands and subsoil in the other case, increases year by year; and perhaps after ten or twenty years, or a longer period, when disaster is threatening the enterprise, pronounced steps have to be taken to correct the trouble and relieve an accumulation that should have been taken care of as a part of the maintenance of the project.

Mr. Robinson and Mr. Mead have rendered a distinct service to the Profession in presenting the data and views on this important topic.

Mr. Post.

GEORGE M. POST,* ESQ. (by letter).—The matter of silt deposition in the Zuni Reservoir has been the subject of a great deal of study, which its seriousness certainly warrants.

When the construction of the dam had been completed, John B. Harper, M. Am. Soc. C. E., estimated, from such information as he had gathered during the years spent there, that the probable life of the reservoir would be thirty-five years. Mr. Robinson, who succeeded Mr. Harper as Engineer in charge of the project, has shown in his paper that the inflow of silt has been even more rapid than was predicted by Mr. Harper.

Several plans looking toward the disposal of this accumulated silt deposit have been proposed, none of which but has one or more insuperable objections. The writer has spent some time on this project and has made a more or less thorough study of the silt question. After having considered the disposal plans offered, he reached the conclusion

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Mr. Post. that the solution of the problem lay in the direction of prevention rather than disposal. In other words, to be, perforce, content with the present capacity of the reservoir and turn attention to the prevention, or at any rate the greatest feasible degree of diminution of the inflow of silt. When search was made for literature on the subject of prevention, it was found that there was almost none, although a great deal has been written about silt disposal.

The topography of the water-shed area of this reservoir is such that the average fall in the watercourses is steep and the resulting velocity is high. The soil, for the most part, is a very fine alluvium, in depth from 5 to 20 ft., most admirably adapted for rapid erosion even by slowly running waters. The majority of the rains on the water-shed are violent showers of short duration, which are also conducive to rapid erosion of the soil. The soil is never found to be moist more than a few inches below the surface. This is due to four causes: First, the annual rainfall is small, perhaps never exceeding 16 in.; second, the steepness of the slopes; third, for a number of years the water-shed area has been overgrazed by the hundreds of sheep and goats which the Indians have pastured there; and, fourth, a combination of the violent character of most of the rainfall and the fineness of the soil. This fourth reason probably requires a little explanation. It is well known that if a sack of flour is dropped into water, an impervious coat of paste will form around the outside and protect the interior for a long time from becoming wet. This same phenomenon occurs on this water-shed. Many times the writer has gone out after one of these storms, lasting from 1 to 3 hours, and has found this almost impervious coat of wet mud, only 2 to 3 in. deep, and perfectly dry soil underneath. This latter condition results in an extremely rapid run-off of the rain-waters, and sudden accumulation of large flows in the main water-courses which scour the stream beds at an extremely high rate.

It is obvious that if some means is adopted to retard the velocity of these rain-waters and hold them longer on the water-shed, the two results most desired would be accomplished. This is the statement of the possible solution, and it is very simple. Perhaps its practical application will be found to be considerably more complicated, and a much more thorough and careful examination of the ground must be made than opportunity as yet has offered. The plan, therefore, can be stated only in general terms.

Practically, the whole area of the water-shed is indented with numerous small gullies, which, running together, form larger ones; these, in turn, form still larger ones, and so on, until the main water-courses are reached. To check the velocity of the water in these gullies requires simple rock and brush structures varying in size and manner of building with the size and shape of the respective gullies. There is an abundance of cedar brush and rock immediately at hand,

and the cost of materials should be low. Care must be taken in the location and construction of these checks. The writer has seen numerous attempts at this sort of work fail by reason of improper location or construction or both. The water must not be allowed to run around the checks and form new gullies. One instance is recalled in which a check of this kind built in a gully to protect a road, simply diverted the water a few feet and now the new gully is larger and deeper than the former one. The checks must be built so as to prevent scouring out deep holes just below them. This difficulty is also of frequent occurrence. The spacing of the checks must also receive careful attention so that the velocity of the water will approach somewhere near that determined upon. This, of course, depends on the slope of the ground in each case. Mr. Post.

The work of construction should not be expensive. Over a water-shed of 659 sq. miles, a large number of these checks, of course, will be required and the total cost will amount to a fairly large figure. One point to be determined is "Will the results warrant the expense?" In the case in hand, the writer's opinion is that they will. Two main benefits will result from this plan. The first and most necessary is the prevention or material diminution of the inflow of silt to the reservoir, and the second, an increase of the absorption of rainfall by the soil. The secondary benefits derived from this last-named item will be increased vegetation and a very considerable increase in the flow of clear water down the stream.

As to whether the first result warrants the expense, the following considerations are the determining factors. The present maximum capacity of the reservoir is 8 290 acre-ft. During 1919 there was deposited in the reservoir a quantity of silt equivalent to 1 250 acre-ft.; should that rate of deposition continue, it will take seven years to fill the reservoir completely and destroy its usefulness.

It cannot be said offhand just what percentage of the erosion will be prevented by the plan proposed. It is, however, conservative to estimate that its quantity will be cut at least in half; and on that assumption, the life of the reservoir can be doubled. When it is considered that the cost of the construction of this dam has been upwards of $300 000 and that because of the construction of the dam, the Indians have been induced to cultivate more than 5 000 acres of land, the success of their crops being dependent almost entirely on the water stored in this reservoir, one would say that prolonging the life of the reservoir only seven years would fully warrant the total probable expense. It is certain that no argument would be needed to convince the Zuni Indians of that.

At the first opportunity, a careful and thorough examination of the water-shed will be made and a cost estimate prepared. It is probable that experience will dictate the construction of a number of different forms of these checks. The writer's idea at present is that most of

Mr. Post. them will be of simple rock and brush construction, built so as to withstand the flow that is likely to come to them, and nothing more.

It is more than probable that the full effects of this checking system will not be apparent for the first two or three years, because there will result a large increase in the absorption of the rainfall by the now dry and thirsty soil and this, as hinted previously, would cause benefits in at least three ways: First, there would be a sure increase in the quantity of vegetation on the water-shed area which itself would assist in further retarding the rainfall, the effect being accumulative; second, the slowly percolating water would eventually reach the reservoir clear instead of being so heavily laden with the destroying silt as is the present condition; third, an equalization of the flow of the stream would result, making more certain the water supply in times when a succession of two or three dry years occurs.

Mr. Robinson. H. F. Robinson,* Assoc. M. Am. Soc. C. E. (by letter).—The writer's paper contained records to the end of 1918. Another year has passed since then and the records for 1919 are now available. (See Tables 4, 5, and 6.)

Soundings were made in January, 1920, and it was found that the silt deposit had continued, and what was more regrettable, that the increase was greater than had been anticipated. It was found that 1 248 acre-ft. of silt had been deposited, while the average annual deposit for the period of 13½ years under consideration was only 557 acre-ft.

A greater quantity might have been expected, for the run-off was 46 473 acre-ft. of water as against an average for the period of 27 648 acre-ft., though observations had led to the conclusion that the average deposit did not depend on the amount of run-off, but was fairly constant. (See Table 4.)

The average silt content of the water during 1919 was 2.66%, or the least percentage recorded except that during 1918, which was the year of the greatest run-off.

The statement was made in the paper (page 875), that:

"It is generally believed that the quantity of silt carried by any stream is largely in proportion to the magnitude of the floods, but, according to the records at Zuni, little relation is found between the magnitude of the flow and the silt carried."

Since this statement was made, the writer has given the matter considerable thought and with the additional information secured during 1919, has decided that his former views were erroneous and that there is a relation between the run-off and the percentage of silt carried by the water. He believes that it may be stated as a general proposition that the percentage of silt carried by the water is in a certain

* Albuquerque, N. Mex.

Mr. Robinson.

TABLE 4.—SILT DEPOSIT IN THE ZUNI RESERVOIR.

All Capacities and Silt Deposits in Acre-Feet.

	Original, 1906.	June, 1910.	January, 1912.	January, 1914.	January, 1918.	January, 1919.	January, 1920.	Total period.
Total silt deposit	0	1 804	3 144	4 200	5 507	6 185	7 433	7 433
Length of period, in years	.....	4	1½	2	4	1	1	13½
Silt deposit for period	.....	1 804	1 340	1 056	1 307	678	1 248	7 433
Silt per year, average	.....	451	894	528	327	678	1 248	557
Total run-off	.....	56 140	24 630	14 369	226 195	5 323	46 173	373 252
Percentage of silt in run-off	.....	3.2	5.4	7.3	0.58	12.5	2.66	1.99
AT CONTOUR 990:								
Capacity	10 230	8 409	7 069	6 013	4 706	4 028	2 801	.....
Percentage remaining of original capacity	.....	82.3	69.2	58.8	46.0	39.3	27.3	.....
Lost capacity per year in percentage of original	.....	4.4	8.8	5.2	3.2	6.7	5.38	.....
AT CONTOUR 1 000:								
Capacity	15 811	13 969	12 632	11 562	10 218	9 540	8 232	.....
Percentage remaining of original capacity	.....	88.4	80 0	73.1	64.6	60.3	52.4	.....
Lost capacity per year in percentage of original	.....	2.9	5.6	3.48	2.1	4.3	3.53	.....

Life of reservoir, Contour 990 = 21 years. Spillway at Contour 991. Capacity between Contours 990 and 991 = 507 acre-ft.
Life of reservoir, Contour 1 000 = 32.4 years.

Mr. Robinson.

inverse ratio to the magnitude of the run-off, within, as yet, undetermined bounds.

In any considerable water-shed it is well understood that any particular run-off may come from only a portion of the entire water-shed, and that the amount of erosion by this particular run-off will depend on the nature of the soil of that locality, so that two separate run-offs of about the same magnitude may carry a different quantity of silt, owing to the different geology and topography of the particular area affected. The varying character of the storm will also affect the rapidity of the run-off and, consequently, the amount of erosion; a long, steady precipitation would cause little erosion while a heavy downpour of short duration would bring down large quantities of silt.

In studying the periods of run-off between the times when the silt deposits have been determined, it has been found that there are six such periods, varying from 5 323 acre-ft. of water for a minimum to the largest one with 226 195 acre-ft., and carrying an average of from 12.5% for the smaller run-off to 0.58% for the greatest. These run-offs with the silt content expressed in percentages have been plotted on cross-section paper, and a tentative curve (Fig. 2) has been drawn, which fits the plotted points fairly well.

TABLE 5.—CAPACITIES, IN ACRE-FEET, OF THE ZUNI RESERVOIR AT VARIOUS TIMES, SHOWING ITS REDUCTION BY SILT DEPOSITS.

Contour.	1906.	June, 1910.	January, 1912.	January, 1914.	January, 1918.	January, 1919.	January, 1920.
940	0	.....	.....	.....	.....	.....	.....
945	7	.....	.....	.....	.....	.....	.....
950	53	.....	.....	.....	.....	.....	.....
955	198	.....	.....	.....	.....	.....	.....
960	591	0	0	.....	.....	.....	.....
965	1 345	185	1	.....	.....	.....	.....
970	2 569	990	249	16	0	.....	.....
975	3 758	2 256	1 190	407	53	12	.....
980	5 563	3 920	2 673	1 720	786	706	99.9
985	7 763	5 997	4 679	3 625	2 330	1 652	1 084.5
990	10 213	8 409	7 069	6 013	4 706	4 028	2 801.2
991	.....	.....	.....	.....	5 213	.....	3 287.4
995	12 887	11 062	9 721	8 664	7 336	6 660	5 410.8
1 000	15 811	13 969	12 632	11 562	10 218	9 540	8 292.1

With a greater number of points to plot along this curve, it is believed that a fairly typical and quite accurate curve might be produced, which would shed considerable light on this question, hitherto scarcely considered.

With the meager data at hand on which to work, these conclusions may be erroneous, but they are submitted for the consideration of the Engineering Profession with the hope that more light may be shed on the subject.

Mr. Robinson.

TABLE 6.—SHOWING RUN-OFF, IN ACRE-FEET, OF THE ZUNI RIVER, AT BLACKROCK, N. MEX.

Total run-off for 1906 and 1907, 38 000 acre-ft.
January to June, 1908, 2 200 acre-ft.
July to September, 1908, 2 900 acre-ft.
October, 1907, to June, 1908, 500 acre-ft.

	1909.	1910.	1911.	1912.	1913.	1914.	1915.	1916.	1917.	1918.	1919.
January	.....	2 540	1 460	.....	.....	1 750	48	11 771	40	40	40
February	.....	.....	1 340	356	.....	2 930	4 604	23 560	712	880	140
March	.....	500	5 200	.....	1 380	5 190	26 695	71 880	1 232	1 240	6 060
April	.....	.....	.....	400	1 420	800	13 737	13 177	354	0	1 040
May	.....	.....	.....	.....	.....	1 200	5 078	55	134	28	.....
June	.....	.....	.....	.....	.....	200	808	.....	98	558	.....
July	1 160	1 800	8 820	1 800	1 380	7 200	1 204	1 444	.....	.....	22 388
August	5 500	2 100	1 660	1 100	300	800	1 859	1 275	600	860	14 726
September	2 840	150	760	.....	3 480	500	740	1 320	505	760	697
October	.....	.....	1 340	380	2 200	450	.....	22 068	102	422	386
November	.....	.....	.....	.....	120	.....	28	38	.....	515	.....
December	.....	.....	.....	.....	50	130	10	.....	.....	20	996
	9 500	7 090	20 580	4 030	10 330	21 150	54 811	146 588	3 777	5 323	46 473

Total run-off, 1906 to 1919, inclusive, 373 252 acre-ft.

Mr.
inson.

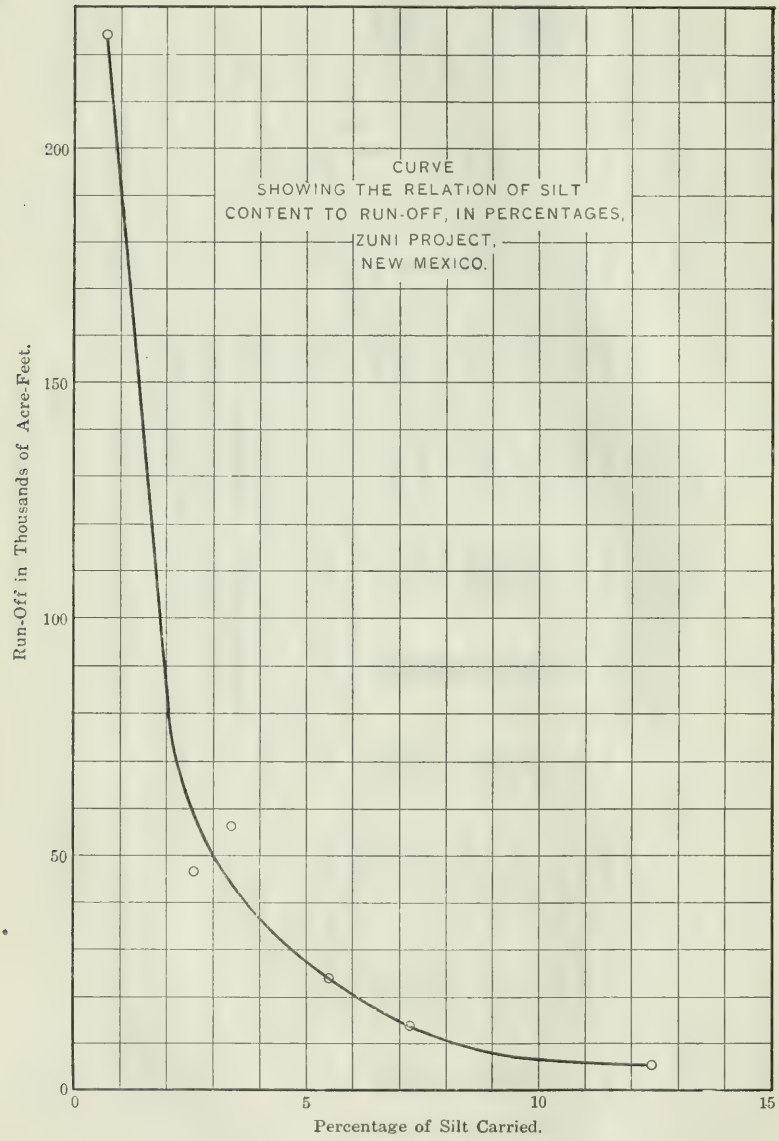

FIG. 2.

Mr. Robinson

The writer has read the discussion by Capt. Jarvis with great interest, and concurs with him in practically all his statements. The point made on the difference between the apparent and actual capacities of reservoirs in countries where the floors and sides of such reservoirs may be of gravel or other loose material and where comparatively large quantities of water may be stored in the interstices, which reappear, as the water recedes, in the form of seeps or springs, is a good one, and has been observed. Unfortunately, however, such a condition does not obtain at the Zuni Reservoir which is situated in a country formed almost entirely of sedimentary stones which on disintegration form a very fine sand or silt; gravel is unknown in that section. With such material, the actual and apparent capacity will coincide closely.

The discussion and suggestions for removal of the sediment are valuable and instructive, but for practice, in this particular case, engineers are again handicapped as there is no waste outflow at a low level, the only openings provided in construction being a number of 15-in. gate-valves in the outlet tower, placed at varying elevations, the lower ones of which have already been buried in the silt deposit.

The writer also concurs in the statement by Capt. Jarvis that the preferable way to prevent excessive sedimentation is to prevent erosion. The valuable discussion presented by Mr. Post, who has looked over the situation on the ground, covers this phase of the question in considerable detail, and the writer desires to offer his thanks for the suggestions presented. The discussions offered by both Capt. Jarvis and Mr. Post are constructive and doubly valuable from that standpoint.

Since the paper was published, some surveys have been made at Zuni, and it is found that it will be perfectly practicable to raise the dam and spillway another 10 ft., which will add perhaps 50% to the life of the reservoir. This will be carrying out to a small extent the suggestion of Mr. Mead that storage reservoirs of this kind be built with a much greater capacity than estimated for the needs of the project. In this case, however, the total life will still be small unless some practicable method of desilting is found.

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Paper No. 1444

THE ACCURACY OF WATER-LEVEL RECORDERS AND INDICATORS OF THE FLOAT TYPE

By J. C. Stevens, M. Am. Soc. C. E.

Synopsis.

The continued advancement of hydraulic engineering is demanding a more extensive use of automatic devices for recording and indicating water levels.

Modern hydro-electric plants are not complete without indicators on the switchboard or devices that record the water levels in forebay, tail-race, net head on the turbines, or levels in some distant reservoir or supply canal. Every water supply investigation involves the recording of water levels in some manner, and no irrigation system can be operated successfully without obtaining and analyzing extensive records of water heights.

This paper is an inquiry into the degree of accuracy that may be expected from the use of automatic recorders and indicators of the float type. It shows that standard instruments now on the market are, in general, sufficiently accurate for most practical purposes, but that where an extraordinary degree of accuracy is required, certain corrections may be applied to the indicated or recorded heights, by which the true heights may be ascertained.

Sources of Error.

In Fig. 1 is shown the general principle on which all float-operated instruments depend. A float rises and falls with the water, which, by means of a line consisting of a cord, cable, tape, or chain, running over a pulley and counterpoised, operates the pen, pencil, indicating hands, or dials of the instrument. In rare cases, a solid rod or rack is used, but the effect is, in general, similar to the use of a counterpoised line.

With a counterpoised line there are three sources of error, which cause the indicated heights to deviate from the true heights:

1. In operating the instrument there is always a certain amount of mechanical work which the float performs. Hence, the indicated heights on a rising stage differ from those on a falling stage by an amount proportioned to the force required to operate the device. This error will be called "float lag."

2. With every change of stage a portion of the line passes from one side of the float pulley to the other. Hence, the indicated heights deviate from the true heights by an amount depending on the range of stage and the unit weight of the line. This error will be called "line shift."

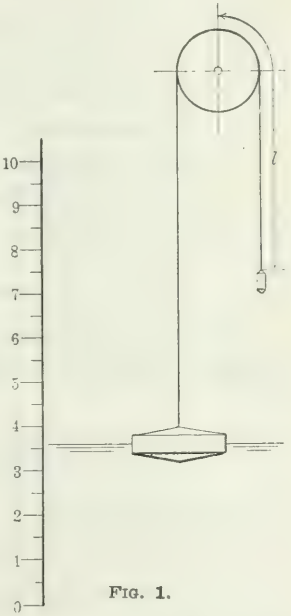

Fig. 1.

3. As the float rises the counterpoise and a portion of the line become submerged, unless provision against submergence is furnished. Hence, the indicated heights deviate from the true heights by an amount depending on the weight and specific gravity of the line and counterpoise. This error will be called "submergence of counterpoise."

In analyzing the effect of these three sources of error, the following notation, in foot and pound units, will be used:

Let W = weight of float.
c = weight of counterpoise.
u = weight per foot of the line.

L = total length of the line.
l = portion of the line on the counterpoise side of the pulley. (See Fig. 1.)
D = diameter of float.
A = area of float.
h = elevation of the liquid or gauge height.
h_1 = gauge height corresponding to submergency of counterpoise.
P = pull of the line on the float.
F = force required to operate the instrument.
w = weight per cubic foot of water or other liquid.
x = depth of flotation of the float with counterpoise in air.
x' = depth of flotation of float with counterpoise submerged.
r and f = subscripts for rising and falling stages, respectively.
s_c = specific gravity of the counterpoise.
s_l = specific gravity of the line.

Correction for Float Lag.

With the float resting on the water and the line attached thereto, as shown in Fig. 1, we have:

$$w\,A\,x = W - P \quad \ldots\ldots (1)$$

on a rising stage, the pull is,

$$P_r = c - F - u\,(L - 2\,l) \quad \ldots\ldots (2)$$

and, on a falling stage,

$$P_f = c + F - u\,(L - 2\,l) \quad \ldots\ldots (3)$$

Substituting Equations (2) and (3) in Equation (1), we have for a rising stage:

$$x_r = \frac{W - c + F + u\,(L - 2\,l)}{w\,A} \quad \ldots\ldots (4)$$

and, for a falling stage,

$$x_f = \frac{W - c - F + u\,(L - 2\,l)}{w\,A} \quad \ldots\ldots (5)$$

Subtracting Equation (5) from Equation (4):

$$x_r - x_f = \frac{2\,F}{w\,A} = \frac{8\,F}{w\,\pi\,D^2} \quad \ldots\ldots (6)$$

Equation (6) gives the lag of the float between the rising and falling stages. It varies directly with the force required to operate

the instrument and inversely as the square of the diameter of the float. To make this error as small as possible, the friction and mechanical work of the instrument should be reduced to a minimum, and the float should be as large as practicable. This lag can never be eliminated entirely.

The force, F, consists of friction in bearings, gears, etc., of the pen or pencil on the paper, work of bending the line over the pulley, and, in the case of long-distance instruments, of making electrical contacts. It will vary with the condition of the instrument as regards oiling, freedom from dirt, etc., and may become inordinately large if the instrument is neglected. It is readily measured by balancing two weights equal to that of the counterpoise on a short piece of line over the pulley. Then, with the aid of a delicate spring balance or scales, the weight required to move the indicator is easily determined.

It has been proven experimentally that if the indicator is set to show the true height on a rising (falling) stage, then the true height on a falling (rising) stage at the same gauge height may be obtained by subtracting (adding) the quantity, $\frac{8\,F}{w\,\pi\,D^2}$, given in Equation (6). With a value of $F = 1$ oz., and an 8-in. float in water, this correction amounts to 0.006 ft.

In general practice, this is a negligible error, but the writer has seen several indicating gauges, where bottles were used for floats, sprocket chains for line, and the counterpoise was an iron index running in grooves over the face of a graduated staff, where this error amounted to 0.25 ft., or more.

It has been found experimentally that the friction factor increases with the weight of the counterpoise; in other words, with the weight on the pulley shaft.

To insure results within a reasonable limit of error, therefore, use a large substantial float, a small counterpoise, a light flexible line, and keep the instrument in good condition.

Correction for Line Shift.

The general equation for depth of flotation of the float is:

$$w\,A\,x = W - c + F + u\,(L - 2\,l) \quad \ldots\ldots\ldots\ldots (7)$$

in which F is + for a rising and — for a falling stage. We may also write:

$$w\ A\ (x + \delta\ x) = W - c \pm F + u\ L - 2\ u\ (l + \delta\ l) \ldots\ldots (8)$$

whence,

$$\delta\ x = -\frac{2\ u}{w\ A}\ \delta\ l;$$

but $\delta\ l$ is the change in stage, that is,

$$\delta\ l = h_1 - h_0$$

whence,

$$\delta\ x = -\frac{2\ u}{w\ A}\ (h_1 - h_0) \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Equation (9) gives the correction for line shift. Obviously, the indicator may be set to show the true height at any gauge height, h_0. Then the correction to be applied is $-\frac{2\ u}{w\ A} H$, where H is the height above or below that point.

For a $\frac{1}{16}$-in., galvanized-iron sash cable, weighing 0.0065 lb. per ft., attached to an 8-in. float in water and a 10-ft. change in stage, this correction amounts to — 0.006 ft. In the case of the indicating gauges previously cited, however, it amounted to nearly 0.2 ft.

It is negative for both rising and falling stages above the origin of H, or the point at which the indicator is set to show the true height, and positive below.

Correction for Submergence of Counterpoise.

When the counterpoise and any portion of the line are submerged, the pull of the float is reduced and, consequently, the depth of its flotation is increased. The weight of any body submerged in water is reduced an amount $\frac{1}{s}$ of its weight in air, where s is its specific gravity. If submerged in any other liquid, s is to be replaced by its specific gravity with respect to that liquid.

When the counterpoise and a portion of the line become submerged, Equation (7) must be written:

$$w\ A\ x' = w \pm F - \left(l - \frac{1}{s_c}\right) c + \left(l - \frac{1}{s_l}\right) u\ (L - 2\ l) \ldots (10)$$

Subtracting this from Equation (7), we have,

$$x - x' = \frac{c}{s_c\ w\ A} - \frac{u\ (L - 2\ l)}{s_l\ w\ A}.$$

Now, at the point of submergence, $l = \frac{1}{2} L$. Consequently, the last term in Equation (10) disappears, and we have,

$$x - x' = \frac{c}{s_c w A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

For a $1\frac{1}{4}$-lb., lead counterpoise and an 8-in. float in water, this correction amounts to 0.005 ft. This correction is always positive and tends to compensate for the error of "line shift."

Correction for Line Shift After Counterpoise is Submerged.

After submergence of the counterpoise, we may obtain a new value of the correction for line shift directly from Equation (9) by writing:

$$\delta x' = \frac{2\left(l - \frac{1}{s_1}\right) w}{w A} - (h_2 - h_1) \quad \ldots\ldots\ldots\ldots\ldots (12)$$

For a $\frac{1}{16}$-in., galvanized-iron sash cord, attached to an 8-in. float in water, this correction amounts to 0.005 ft. for a 10-ft. change in stage, or 0.001 ft. less than for the counterpoise running in air.

Correction Diagrams.

The foregoing corrections are best applied in practice by preparing a correction diagram from which the combined corrections may be applied to the indicated heights to ascertain the true heights.

It is to be remembered that most instruments in good condition, if properly installed, will indicate or record the true height within a reasonable limit of error. It is only in cases where extraordinary accuracy is desired that these corrections need be applied. For example, in making tests of pumping plants where discharges are fluctuating and it is desired to secure continuous records of the head on a discharge measuring weir to within 0.001 ft.; or, again, if it is desired to measure the evaporation from a tank or reservoir, or the quantity of oil pumped into and withdrawn from a tank. In such cases, continuous records are required to a much greater degree of accuracy than in the ordinary practice of hydrometry.

Wherever such exigencies exist, they can be met by the use of ordinary recording instruments by applying the necessary corrections.

In order to prepare a correction diagram, it is necessary first to determine the constants of the instrument and its installation.

A prime requisite for all installations is a stilling chamber in which the float may move without being influenced by wave action, and where the line may be protected from wind.

Once installed, it is necessary to measure:

1. The diameter of the float;
2. The weight of the counterpoise and line;
3. The gauge height at which the counterpoise is submerged; and,
4. The force, in pounds, required to move the indicator.

To illustrate, a numerical example will be taken: The float has a diameter of 8 in. and weighs about 4 lb. The counterpoise is of lead, weighs 1¼ lb., is 0.2 ft. long, and is submerged at a gauge height of 5.0 ft. The line is $\frac{1}{16}$-in., galvanized-iron sash cord, weighing 0.0065

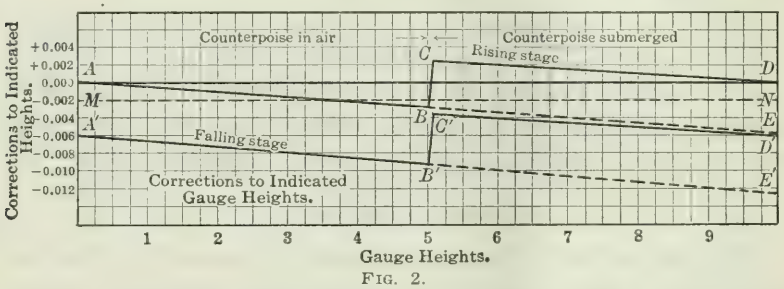

FIG. 2.

lb. per ft., and has a specific gravity of 6, since it has a hemp center. The force required to move the recorder pencil is 1 oz.

Substituting these values in the equations previously given, the following corrections are obtained:

1. For "float lag", Equation (6), the correction between a rising and falling stage is 0.006 ft.

2. Correction for "line shift" with counterpoise in air, Equation (9), is $-\ 0.0006\ (h_1 - h_0)$ ft.

3. Correction for "line shift" with counterpoise submerged, Equation (12), is $-\ 0.0005\ (h_2 - h_1)$ ft.

4. Correction for submergence of counterpoise, Equation (11), is + 0.005 feet.

We have two correction lines—one for a rising stage and another exactly parallel at a distance of 0.006 ft. for the falling stage. These corrections are plotted in Fig. 2. The corrections for a rising stage are given by the line, *ABCD*, and those for the falling stage by

A′ B′ C′ D′, which is parallel thereto and 0.006 ft. below. The lines, *AB* and *A′ B′*, have a slope of 0.0006 ft. per ft., and the lines, *CD* and *C′ D′*, have a slope of 0.0005 ft. per ft. The lines, *BC* and *B′ C′*, are the correction of 0.005 ft. for submergence of the counterpoise, which occurs in 0.1 ft. of gauge height.

If the counterpoise is run in a pipe from which the water is excluded, then the lines, *AE* and *A′ E′*, represent corrections.

These lines are plotted on the assumptions that the pencil is set to record the true height at zero gauge height on a rising stage. In practice, the pencil must be set to record the true height at whatever height the water happens to be.

In order to obviate the necessity of constructing a new correction diagram for each setting, the correction lines may be drawn on a sheet of tracing paper. Then, by shifting this sheet up or down over the scale of gauge heights, the corrections for any setting are readily obtained. For example, set the pencil so that it indicates the true height on a rising stage at a gauge height of 3.5 ft. Then, shift the tracing paper until the correction line, *ABCD*, for a rising stage, intersects the gauge height line of 3.5 ft. at the origin of corrections. This is equivalent to moving the origin of corrections to the line, *MN*, in Fig. 2.

Any instrument may be set on the rising stage by pulling on the line on the side of the float until it is slack below the hand, then slowly releasing it.

It is seen from Fig. 2 that the maximum corrections are less when the counterpoise is allowed to submerge, than when run in a sealed pipe.

Instruments Performing Mechanical Work.

In some types of instruments a certain amount of mechanical work is done in addition to that of overcoming friction. For example, in long-distance recorders, electrical contacts are made at stated intervals of gauge height.

The pen remains stationary until the float has traveled the amount of the contact interval, then it suddenly follows the float that amount. The pen should nominally indicate the true height within an amount equal to the contact interval. However, if the pen is set to indicate the true height just after a contact on a rising stage, it will deviate from the true height just before a contact on the falling stage by an

amount equal to the contact interval plus the float lag due to friction alone.

In Fig. 3 are plotted the results of an actual test on a long-distance recorder. This instrument was one of the manufacturer's earlier types and had a nominal contact interval of 0.05 ft. (actually 0.0479). It was tested with a 10-in. float in a tank in which water and float heights were measured to 0.001 ft.

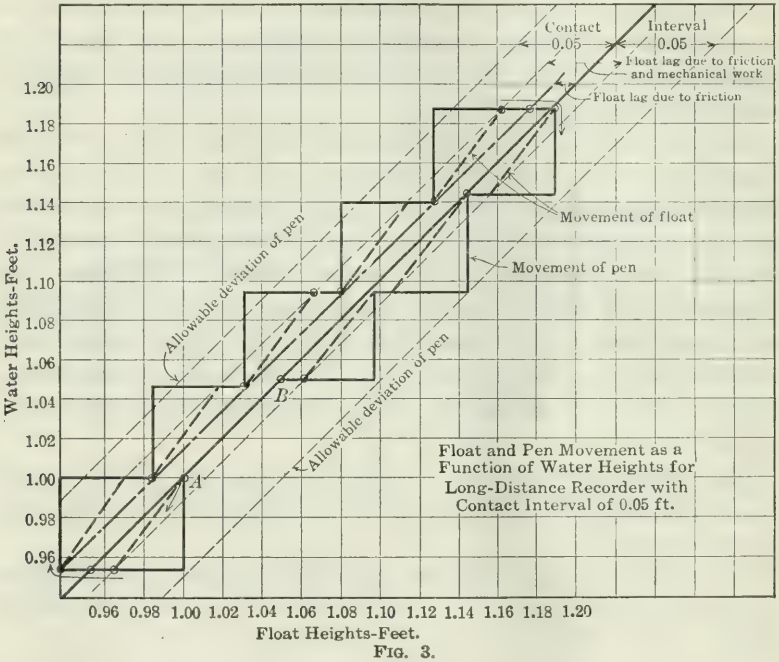

FIG. 3.

The pen was set just after a contact on a falling stage to indicate the true height, at the point *A*. The water was lowered, then raised, and lowered again to the point *B*. The float and pen movements are plotted as functions of water heights.

The float lag due to friction alone was 0.012 ft., and the lag due to friction and mechanical work was 0.036 ft. The pen, however, indicated the true height within the allowable deviation of 0.05 ft., except just before contact on the rising stage, when it exceeded the allowable deviation the amount of the friction lag 0.012 ft.

This error was inappreciable for this type of instrument, but could easily have been reduced by increasing the size of the float.

Similarly, in Fig. 4, are plotted the results of a test on a long-distance indicator built by the same manufacturers. This instrument had a contact interval of 0.01 ft., and the pen was to indicate the true height within that limit of error. During the test, the deviation was within that limit on the rising stage. On the falling stage, just the instant before contact, this limit was exceeded the amount of the friction lag 0.0007 ft. A 24-in. float was used. The instrument had a total range of 35 ft., but the maximum error due to line shift and submergence of counterpoise was less than 0.002 ft.

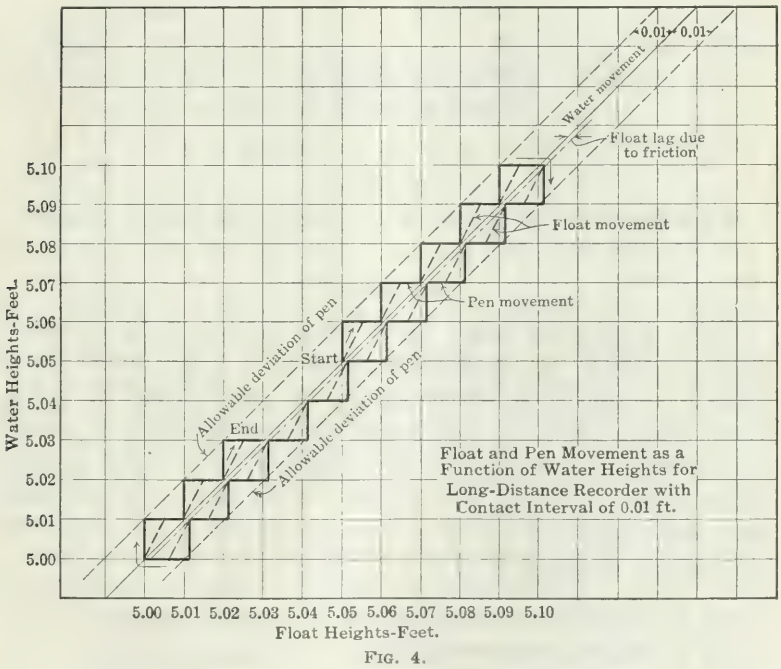

FIG. 4.

In conclusion, it is well to repeat that most recording and indicating instruments are well within the practicable limits of required accuracy, if they are kept in good condition. The best results are secured by using a large substantial float, a light flexible line, and as small a counterpoise as possible, allowing it to submerge at mid-range of gauge heights. Where greater accuracy is required, a correction diagram is readily prepared by which the true heights may be obtained from the recorded heights.

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Paper No. 1445

THE PRODUCTION OF TOLUOL FROM GAS PLANTS

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With Discussion by Messrs. A. C. Klein and Horace C. Porter.

Synopsis.

Toluol is the hydro-carbon which forms the base of the well-known explosive, T.N.T. In 1914, the production of toluol in the United States was estimated at 700 000 gal. for the year; during the war the United States Ordnance Department undertook measures to increase the production and, at the time of the signing of the armistice, it was conservatively estimated that, at the end of 1919, such production would have been at a rate of at least 32 000 000 gal. per year. This increase was obtained:

(*a*) By stripping gas manufactured in water-gas plants;
(*b*) By building coke oven plants;
(*c*) By cracking petroleum oils and solvent naphtha; and
(*d*) In various other ways yielding small quantities.

The development, with regard to stripping water gas, furnished interesting problems in connection with the standards of illuminating gas established by Public Service Commissions throughout the country and, for this reason, became involved in the National problem of the conservation of fuel. In New York City, the situation was further

complicated by State legislation which regulated not only the price of gas, but its quality.

This paper traces these complications, and brings forward for consideration the question, whether or not an enormous waste in fuel is still taking place, because of heat or light standards established when it was comparatively cheap.

At the beginning of the war the standard United States Army high explosive was Trinitrotoluol, more generally known as T.N.T., which is obtained by nitrating toluol, a colorless, oily liquid. Chemically, toluol is a hydrocarbon of definite composition, having the formula, $C_6H_5CH_3$, of the so-called benzene series. The specifications of the United States Ordnance Department require that its paraffin content shall not be more than 3% of the volume, and that it shall all distil between 109.4° and 111.4° cent. About 15 lb. of crude T.N.T., or 13½ lb. of refined T.N.T., can be produced from 1 gal. of toluol.

Prior to August, 1914, toluol was made in the United States to a limited extent, the production in that year being estimated at about 700 000 gal. It was used in chemical trades, as, for example, in the rubber industry and in the manufacture of saccharin, drugs, and dyes, but, for the most part, it was not separated from other oils with which it was manufactured. At that time, the principal source of toluol was from the by-product coke ovens, where it was obtained from the distillation of coal, in specially designed condensing apparatus; small quantities were also obtained from some of the gas companies from their gas-holders and drips.

Shortly after the beginning of the war, in 1914, both the British and French Governments desired great quantities of T.N.T., and, in consequence, many toluol and T.N.T. plants were placed under construction in the United States. By the spring of 1916, the capacity of the T.N.T. plants in this country had reached an output of about 5 000 000 lb. per month, and plants of less capacity were operating in Canada. Practically all the toluol needed for these plants was produced in the United States, and, in addition, some excess was produced and exported for nitration to British and French factories.

The urgent demand for toluol resulted in forcing its price to $7.00 per gal., in the spring of 1915. The price fluctuated between that and $4.00 per gal., until the spring of 1917, although trade journals have

recorded sales as high as $12.00 per gal. In April, 1917, the market price was $4.50 per gal.

It was apparent in May, 1917, when the requirements of the Ordnance Departments of the Army and Navy took definite shape, that there would be a most serious shortage in toluol. In June, a conference between these Departments, as well as representatives of England, France, Italy, and Russia, was held, and in July, 1917, the Council of National Defense appointed a board to investigate this situation. This Committee reported on July 27th, 1917, that the existing plants would not be able to meet the demand. A Sub-Committee on Coal Tar By-Products was formed and instructed to report as to methods to be followed to relieve the situation, and it was not until September 17th, 1917, that its report, with recommendations, appeared.

It is apparent, consequently, that, at the beginning of the war, the United States was not prepared to meet the situation with regard to its most important high explosive. Fortunately, the requirements of T.N.T. were considerably decreased in the fall of 1917 by the adoption by the Ordnance Department of amatol as the standard shell-filler for explosive shells. Amatol is a mixture of T.N.T. and Ammonium Nitrate, in proportions varying from 20% of T.N.T. and 80% of Ammonium Nitrate, to 50% of either, these mixtures being used at that time by the Allies as one of their standard high explosives.

The Gun Division of the Ordnance Department was charged with the responsibility of relieving the situation with regard to toluol, and Col. Jay E. Hoffer, then in charge, assisted by Col. (then Maj.) James H. Burns, requested permission from the War Industries Board, in September, 1917, to contract with gas companies for the construction and operation of toluol-recovery apparatus. There was formed, therefore, within the Gun Division, the Toluol Branch of the Explosives Section, which took complete charge of the constructive development of the toluol industry. Steps were taken immediately to investigate every possible source from which toluol could be derived, but, inasmuch as a definite recommendation had already been made to build apparatus in the works of gas companies to strip domestic gas of its light oil content, the first measures adopted were to negotiate contracts for the construction of such apparatus and to obtain the consent of the various gas companies and Public Service Commissions interested, so that this work could proceed.

These negotiations were most difficult, because stripping the gas decreased its candle-power. The question of the candle-power furnished by the gas companies to consumers had been for years a most delicate one, and, therefore, representations had first to be made to the Public Service Commissions requesting that the standard of candle-power be reduced. Amicable negotiations were concluded between the Ordnance Department and the Public Service Commissions in all cities in which it was desired to make such installations, except in the case of New York City, where the Public Service Commission laid down rules impossible for the gas company or the Ordnance Department to accept. The urgency of the situation in the case of New York City required drastic action, inasmuch as this city was expected to furnish a very large portion of the toluol. With the acquiescence of the gas companies it was determined, therefore, to proceed immediately to construct within the plants of the Consolidated Gas Company and the Brooklyn Union Gas Company thirteen stripping plants for the extraction of toluol, the Ordnance Department guaranteeing, for its part, to hold harmless the gas companies from any damages whatever caused either by the construction or operation of such plants, and the gas companies, on their part, agreeing to furnish the land free of charge and to operate the plants at absolute net cost, without any profit whatever.

Altogether, thirty-two gas-stripping plants were placed under construction in all parts of the country, from Boston, Mass., to Albany, N. Y., to Washington, D. C., to New Orleans, La., St. Louis, Mo., Denver, Colo., and Seattle, Wash., and contracts for the construction of all these plants were let as promptly as possible to three contractors, the work being divided between them as the judgment of the Toluol Branch deemed best under the circumstances.

It should not be thought, however, that the choice of location of these various plants was left to any haphazard guess. The Toluol Branch was very careful prior to awarding any contract to make tests of the gas that was being manufactured, to determine the suitability of its toluol content for nitration. Estimates of cost of construction and operation were gone into most carefully, the general rule laid down having been that no plant should be undertaken which would cost for construction more than the product obtained by multiplying its estimated yearly output, in gallons, by $2.00, and unless it could be operated at an expense not to exceed $1.50 per gal. of output.

Provision had also to be made for refining the crude toluol which was to be obtained from these gas plants, and for transporting both the crude and pure toluol, 250 tank cars having been ordered and purchased for this purpose alone. In addition, arrangements had to be made for the chemical control of the plants, to devise test methods for operation, and to instruct and drill the operators.

The gas which was stripped by the Ordnance Department, was usually carbureted water gas, manufactured by adding gas-oil vapors to a blue gas formed by the chemical interaction between steam and incandescent coke or hard coal. As a rule, from 3 to 4½ gal. of oil were added to each 1 000 cu. ft. of gas manufactured, the quantity of oil used being dependent on the illuminating quality and heat units required in the gas as finally sent out. In New York City, for example, the State laws required the standard for gas to be 22 c-p., and, for this purpose, generally, 4½ gal. of gas oil were required for each 1 000 ft. of gas. This gas contained approximately 650 B.t.u., or more, per cu. ft., and the gas-stripping apparatus through which it passed recovered as toluol about 2% of the gas oil used in the manufacture of the gas. In addition, however, to the toluol recovered, a considerable quantity of benzol and solvent naphtha was also recovered, the quantity of benzol being about one and one-half times that of the toluol and the quantity of solvent naphtha about one-half that of the toluol. These other by-products were also valuable for explosives, inasmuch as benzol was used as the base for picric acid, the favorite high explosive of the French, and solvent naphtha was in demand by the Navy for one of its high explosives—tri-nitro-xylol—although the manufacture of the latter by the Navy never reached quantity production. Large quantities of picric acid, however, were made in the United States for the French, particularly as a replacement to them of explosives loaned to this Government.

Toluol Recovery.

The gas, as manufactured by the gas companies, is passed through a tall tower filled with wooden grids through which an absorbent oil is circulated. The gas passes counter-current to this absorbent oil and loses its toluol, benzol, and xylol content, and is then distributed through the mains of the gas company to the consumers, without these light oils. The quantity of these light oils which can be absorbed, is

dependent both on the quantity of absorbent oil circulated through this tower, on its absorbing quality, and on the temperature and the velocity of the passing gases, so that wide variations of stripping can be obtained by manipulation. Later, the light oils are separated from the absorbent oil by the customary distillation processes using steam.

The gas which is then distributed through the mains, has lost a considerable portion of its candle-power, as well as some of its heat content. It is possible in some cases to add enough gas oil in the original manufacture of the gas so that, even after scrubbing, the gas will have the desired candle-power, but this method of retaining candle-power is most extravagant and wasteful, since oil is the most expensive ingredient used in the manufacture of gas; or, the gas may have its former lighting power restored by spraying back into the scrubbed gas other light oils, exclusive of toluol, but this, naturally, is also most extravagant, because these light oils have a distinct commercial value.

New York City.—In New York City, the Public Service Commission had made an official order insisting either that the full statutory 22 c-p. be furnished to the consumer, or that the gas company, not exceeding the period of the war, deliver gas containing 650 B.t.u. per cu. ft., with rebates whenever the gas dropped below this standard, and requiring, further, in the latter case, that each consumer using flat flame burners, on application, would be furnished with two incandescent mantle burners, and requiring, further, that the gas companies maintain a number of testing stations.

The gas companies refused to accept this permissive order of the Public Service Commission, and matters with regard to the operation of these plants made no progress, although their construction was actively pushed, and the 21st Street Plant of the Consolidated Gas Company, one of the largest of its plants, was ready for operation on April 14th, 1918.

Officials of the Ordnance Department had held frequent conferences with the Public Service Commission, but the Commission still held its ground that the consumers of New York City must be furnished either with gas of 22 c-p., or with gas fulfilling all the requirements of its order, and that, if necessary, either the Government would have to furnish additional gas oil to the companies, if the standard could be maintained by that method, or measures of operation would have to be adopted which would meet the order.

At this point there arose a further complication; the winter had been most severe, and the movement of all fuel supplies was extremely slow. It may even be remembered that the Fuel Administration during this time had ordered fuelless days, and it was absolutely out of the question to obtain for the gas companies more oil than they were using at that time. It was most difficult for them to obtain any oil at all, inasmuch as cargoes of it were frequently commandeered by the Army and Navy for direct military operations.

At a final conference held early in April, this situation was again explained by the Ordnance Department to the Public Service Commission, and the crying need for toluol was made clear. Under these circumstances, the Public Service Commission finally consented to permit the Ordnance Department to operate these gas-stripping plants as it deemed proper under the circumstances, it being suggested at that time that the gas to be sent to consumers should not fall below 18 c-p. and that careful records would be kept as to the complaints by consumers and as to additional quantities to be used by them.

By order of the Ordnance Department, the 21st Street Plant, therefore, was placed in operation on April 14th, 1918, followed in due course by the other plants. It was found that the consumers did not notice the difference in the quality of the gas, and experiments were carried out, therefore, at the Mutual Gas Company's plant, to determine whether further lowering would cause any complaints. The candle-power was dropped gradually at this plant from 18 c-p. to as low as 10 c-p., and yet no additional complaints were received from the consumers. When this situation was further explained to the Public Service Commission, the Commission acquiesced in permitting the representatives of the Ordnance Department to operate these plants at a standard lower than the 18 c-p. originally set, and, in consequence, a gradual decrease in candle-power output was effected at all plants until, at the time of the signing of the armistice, the average candle-power in New York City was approximately 16, and yet the consumers were apparently ignorant of the fact.

Careful records were also kept as to the loss in heat units resulting from the stripping of the gas, and it was curious that no exact ratio could be found between loss of candle-power and loss of heat units. It is probable that the average heat units in a cubic foot of gas which

averages 16 c-p., were in the neighborhood of 620 B.t.u.; in other words, the gas lost 27% of its illuminating power and less than 5% of its heating power.

Mention has been made of the serious shortage in the supply of oil during the spring and summer of 1918. The distribution of oil throughout the United States had been placed in the hands of the Fuel Administration, and one of its assistants conceived the idea that a tremendous quantity of oil could be saved for the country's commercial industries for war purposes if the gas companies throughout the country were prohibited from using their usual supply of gas oil. The Fuel Administration had taken a census of the total quantity of oil used by the gas companies for this purpose, and had estimated that the average heating value of the gas produced throughout the United States was about 600 B.t.u. per cu. ft. of gas, and that if a compulsory order were issued by the National Government ordering that no gas be furnished to consumers higher than 528 B.t.u. per cu. ft., that a possible saving would be made of about 0.7 gal. of oil per 1 000 cu. ft. of gas. This proposed action of the Fuel Administration was of the greatest importance to the Toluol Branch, which at that time had practically completed plants estimated to produce 5 000 000 gal. of toluol per year, and as the production of toluol was directly dependent on the quantity of gas used, this order would have resulted in a loss of production of 1 500 000 gal. of toluol per year from the Ordnance plants alone, and a loss of about 750 000 gal. from plants owned by private interests. The Ordnance Department, however, studied the problem, and obtained figures to show that the large savings in oil by such a reduction in standard were not as great as the Fuel Administrator believed, and efforts were made to impress the importance of non-interference with the production of toluol.

On April 23d, 1918, the Fuel Administration was preparing to issue an order forcing the gas companies to cut down their consumption of oil to 2¾ gal. per 1 000 cu. ft. of gas manufactured. It should be noted that this action of the Fuel Administration would naturally have the backing of all gas companies using gas oil in manufacture, as it would enable them to reduce their operating costs by a material amount and would possibly give them renewed hearings before Public Service Commissions for a re-opening of rate cases, which many of the companies,

because of increased costs, were then endeavoring to obtain. The Ordnance Department persuaded the Fuel Administrator not to issue this order (see Appendix I) without giving all the departments interested a chance to appear, and, on May 18th, 1918, a conference was held at the office of the National Committee on Gas and Electric Service. Many reports and statistics had been prepared by the gas industry to support the order of the Fuel Administration; in general, all gas company officials testified that the consumer received the same benefit from gas of low heating value as from gas of high heating value, while representatives of the Public Service Commissions and of the Bureau of Standards testified to the contrary. The Ordnance Department at no time took sides in the dispute as to the merits of this phase, as it recognized that such matters were not within its jurisdiction, but always based its arguments against issuing the order, because of its effect on the toluol programme, on two distinct reasons: (*a*), that the quantity of oil saved would not be great, inasmuch as the majority of gas plants would still require the same quantity of oil in order to produce toluol; and (*b*), in that case, the oil required in excess of that needed to furnish 528 B.t.u. gas, would have to be furnished to the gas companies by the Ordnance Department at an expense to it of many millions of dollars.

The Toluol Branch prepared statistics (Table 1) showing the production of toluol from gas plants, assuming the standards to remain unchanged and then to be established at 575, 550, and 528 B.t.u. per cu. ft. of gas, and also statistics (Table 2) showing the total quantity of gas oil used in the New England, Middle Atlantic, Middle West, Southern, and Pacific States, both in plants producing toluol and those not producing toluol.

On June 5th, 1918, a conference was held at the office of the Fuel Administration, at which representatives of the various State Public Service Commissions presented their views. On the following day, the Oil Administrator, and representatives of the Ordnance Department, discussed all the statistics that had been prepared, and, as a result of this conference, the Oil Administrator stated that he was convinced that the quantity of gas oil saved by issuing an order for a lower standard for gas would not warrant the order.

Consequently, the Ordnance Department considered the matter closed, but, apparently, interests connected with the gas industry were

TABLE 1.

Production of Toluol from gas plants, in gallons.		Reduction in production, in gallons.
With existing standards.....	9 400 000	.....
" 575 B. t. u. "	8 450 000	950 000
" 550 " " " "	8 000 000	1 400 000
" 528 " " " "	7 500 000	1 900 000

TABLE 2.

	Gas Oil Used, in Gallons:		Total, in gallons.
	In Toluol Plants.	In Non-Toluol Plants.	
New England States.....	48 750 000	18 000 000	66 750 000
Middle Atlantic "	426 902 000	156 000 000	583 702 000
Middle West "	164 053 000	60 600 000	224 653 000
Southern "	9 700 000	3 580 000	13 280 000
Pacific "	2 600 000	961 000	3 561 000
	652 005 000	239 941 000	891 946 000

Note.—Oil-gas process plants on the Pacific Coast are not included.

not satisfied. The Fuel Administration announced, about the end of June, that a Director of Gas Plants had been appointed, and the question of lowering the standard was immediately re-opened. On July 22d, 1918, the Director of Gas Plants called a meeting at his office, at which there were present representatives of the Fuel Administration, the Bureau of Standards, the War Trade Board, the National Committee on Gas and Electric Service, and the Ordnance Department. The Director of Gas Plants was very frank in announcing that it was his intention to have the Fuel Administration issue a notice ordering the standard of all artificial gas to be 528 B.t.u. per cu. ft. The Ordnance Department immediately took exception to the order as prepared (see Appendix II), basing its opposition on two broad claims: First, that the order as issued would save very little oil, and, second, that it would transfer millions of dollars from the Treasury of the United States to the gas companies. Because of these objections, the Director postponed issuing this order, and a further conference was called for August 28th, 1918, at which representatives of the gas companies, the Public Service Commissions, the Fuel Administration, the Bureau of Standards, War Industries Board, War Trade Board,

and Ordnance Department, were present. At this meeting only representatives of the gas companies were heard, and they were almost unanimous in stating that, in their opinion, the consumers would not feel the reduction in heat or light standards and that it was for the best interests of the country to reduce these standards. It seemed to the representatives of the Ordnance Department that the Director of Gas Plants had definitely determined to issue the order, and the matter was deemed so urgent that the Ordnance Department presented its case through the Chief of Ordnance to the Secretary of War.

In consequence, on September 7th, 1918, the Assistant Secretary of War addressed a letter (see Appendix III) to the Fuel Administrator citing the history of the case and requesting that the so-called 528 B.t.u. order be not issued. On receipt of this letter, the Fuel Administrator appointed a new committee consisting of the Director of Gas Plants, the Director of Natural Gas Plants, and a representative of the Bureau of Standards, to report as to the advisability of issuing this order. This Committee presented a divided report, the representative of the Bureau of Standards recommending that the order be not issued. The Committee finally agreed, however, as a compromise, that in place of issuing a mandatory order to the various gas companies throughout the country, a request (see Appendix IV) be made by the Fuel Administrator to local authorities throughout the country that they arrange with their service corporations to maintain as low a standard as could be considered consistent with efficient and good service, the object being to conserve the maximum quantity of fuel for more necessary purposes. The form of this request was satisfactory to the Ordnance Department, inasmuch as it left open to that Department a chance for appeal to local authorities should it be found that the toluol programme would be affected adversely.

The signing of the armistice, on November 11th, 1918, changed conditions again, and, shortly after that time, Dr. Harry A. Garfield issued the following notice to the press:

"U. S. Fuel Administration.

"Washington, D. C.

"B. T. U. Standard Not To Be Changed.

"United States Fuel Administrator, Harry A. Garfield, announced to-day that, under existing conditions, he did not feel warranted in

recommending to local boards the abandonment of existing gas standards as a fuel-saving measure.

"A special committee was appointed in connection with the Bureau of Standards to investigate the heat value of artificial gas, but since the creation of this committee, Mr. Garfield pointed out, the fuel situation has materially changed.

"Public Service corporations are asked to consider whether a reduction of British Thermal Unit Standards by companies under their jurisdiction, will actually save oil, with the recommendation that if a reduction in quality is ordered, there should be simultaneously a corresponding reduction in the price of gas. The statement says:

"'In July last the United States Fuel Administrator appointed a special committee in conjunction with the Bureau of Standards, to investigate the heat value of artificial gas with a view to recommending what change, if any, in the B.t.u. standard would save coal or oil. Since the creation of this committee, the fuel situation has materially changed. The production of bituminous coal has increased under the stimulus of the Administration to unheard of figures, and the situation is not as critical as last summer. Yet, it is probable that under zoning orders of the Fuel Administration, some gas plants throughout the country will not be able to receive their customary rate of coal, and this may necessitate a temporary reduction in gas standards.

"'At the cessation of hostilities the demand for oil and coal will be transferred to other channels and the pressing demand for immediate deliveries will be reduced to such an extent that the Fuel Administrator does not feel warranted in asking local boards to abandon existing standards.

"'While it is not within the province of the Fuel Administration to request thrift in general, Mr. Garfield desires to call attention to the fact that we are using our oil deposits perilously fast, and therefore Public Service Commissions should consider whether or not a reduction of B.t.u. standards by the companies under their jurisdiction will actually save oil. In the interest of the public, however, wherever a reduction in quality is ordered or approved, there should be simultaneously made a corresponding reduction in the price of gas.'"

Finally, it is most interesting to point out what the Ordnance Department did accomplish between May, 1917, and October, 1918. Table 3 shows that the production was more than doubled in that period; and it also shows to whom the supply was distributed. It should be remembered, however, that the work of increasing the supply was still under way at the time of the signing of the armistice; grant-

TABLE 3.

Quantity, in gallons.	Source.		Quantity, in gallons.
In May, 1917:			
690 000	Coke ovens	of which	200 000 was taken by Allies as raw toluol.
75 000	Private gas plants		230 000 " " " " , nitrated.
.....	Ordnance plants		330 000 " " " Navy.
35 000	Miscellaneous		40 000 " " " Chemical Industry.
800 000			800 000
In October, 1918:			
1 036 000	Coke ovens	of which	270 000 was taken by Allies, nitrated.
270 000	Private gas plants		100 000 " " " Navy.
337 000	Ordnance plants		82 000 " " " Chemical Industry.
54 000	Miscellaneous		925 000 " " " Army, nitrated.
			320 000 " surplus for Army.
1 697 000			1 697 000

ing no unforeseen contingencies, the production would have reached 2 861 000 gal. per month by the middle of 1919.

APPENDIX I.

"UNITED STATES FUEL ADMINISTRATION
"OIL DIVISION

"WASHINGTON, D. C., MAY 17TH, 1918.

"ORDER.

"M. L. REQUA, *Director.*

"It appearing to the United States Fuel Administrator that it is essential to the National security and defense, and the successful prosecution of the war, that the greatest possible use be made of our supplies of petroleum,

"The United States Fuel Administration, acting under authority of an Executive Order of the President of the United States, dated 23 August, 1917, appointing said Administrator, and of subsequent Executive Orders, and, in furtherance of the purpose of said Order and of the Act of Congress therein referred to and approved August 10, 1917.

"Hereby orders and directs that until further or other order of the United States Fuel Administrator,

"The standard for artificial gas as manufactured for distribution, is hereby fixed at 528 British Thermal Units per cubic foot (Massachusetts standard), superseding all previous standards.

"The above order is to take effect immediately and to become operative with the least possible delay.

"Nevertheless, it is expressly understood that all gas companies, with toluol plants, shall use sufficient oil and shall conduct their operations in accordance with the requirements of the Ordnance Department, so as to obtain the maximum amount of toluol which their plants can produce; but the B.t.u.'s in the distributed gas, must not be reduced below the standard (of 528) fixed herein, without special license from the Ordnance Department.

"UNITED STATES FUEL ADMINISTRATOR."

APPENDIX II.

PROPOSED ORDER BY JAMES T. LYNN, DIRECTOR OF GAS PLANTS, JULY 22D, 1918.

"ORDER.

"1. It appearing to the United States Fuel Administrator that it is essential to the National Security and Defense, and the successful prosecution of the war, that the greatest possible use be made of our supplies of petroleum and coal.

"2. The United States Fuel Administrator, acting under authority of an Executive Order of the President of the United States, dated

23 of August, 1917, appointing said Administrator, and of subsequent Executive Orders, and in furtherance of the purpose of said Order and of the Act of Congress therein referred to and approved August 10, 1917.

"3. Hereby orders and directs until further or other orders of the United States Fuel Administrator:

"4. That in all plants in which artificial gas is produced for public consumption, and in which toluol or toluol-producing material is not recovered for the United States Government, or its contractors or agents, the use of gas oils and bituminous coal is hereby restricted to that amount necessary in each of said plants to produce gas having a gross heating value of five hundred and twenty-eight (528) British Thermal Units per cubic foot delivered to the consumer as hereinafter specified.

"5. That in artificial gas producing plants which now, or at any time hereafter, while this order is in force, recover toluol or toluol producing material for the United States Government, its contractors or agents, the use of gas oil is hereby permitted in excess of the amount specified in Clause 4 of this order, for the purpose of recovering such amounts of toluol or toluol-producing material, as is or shall be required in each case by the United States Government, its contractors or agents.

"6. That all illuminating-power standards of quality for artificial gas supplied for public consumption now in effect are hereby suspended.

"7. That heating value standards of quality in excess of five hundred and twenty-eight (528) British Thermal Units per cubic foot for artificial gas supplied for public consumption, are hereby suspended.

"8. That the requirement of heating value herein made shall be deemed to be fulfilled if the gas tested at any point within a radius of one mile from the manufacturing plant, shall have an average gross heating value of not less than five hundred and twenty-eight (528) British Thermal Units per cubic foot as determined from the monthly average of daily tests, and if the gross heating value of the gas so tested, in excess of this amount, shall be kept to a minimum as closely as the condition in each case shall permit.

"9. That this order shall be in effect on the fifteenth day of August, 1918, or within a period of thirty days thereafter, if the conditions in any artificial gas plant or the territory served by it require this time before the order goes into effect, but in no case shall the non-observance of this order be permitted beyond the 30-day period herein provided except by special permission of the Fuel Administrator.

"UNITED STATES FUEL ADMINISTRATOR."

APPENDIX III.

"SEPTEMBER 7, 1918.

"MY DEAR MR. GARFIELD:

"1. It is understood that the Fuel Administration is considering the issue of an order establishing throughout the country a fixed standard for artificial gas of 528 B.t.u. per cubic foot, as a measure of conserving oil and coal.

"2. I desire to bring to your attention the effect which such an order will have upon the production of toluol from gas scrubbing plants.

"3. As one means of meeting the serious shortage in the supply of toluol required for the Army Programme, special efforts have been made to recover the maximum amount of toluol from water gas plants. The Ordnance Department is spending over $7 000 000 for additional plant facilities of this character, and from the gas scrubbing plants so provided, plus those installed by private corporations, it is estimated that toluol will be recovered from 65% of the water gas produced in the country. The yield which the Ordnance Department has been counting on is predicated on the continuance of the present heat standards.

"4. If a 528 B.t.u. order goes into effect, it seems reasonable to assume that in order to maintain the required toluol production, the Ordnance Department will have to make arrangements to supply the oil at the gas stripping plants corresponding to the difference between the requirements to maintain the present standards and the proposed standard of 528 B.t.u.

"5. To this extent, it is believed that there will be no conservation of gas oil and the Ordnance Department estimates that they will be put to an extra and unexpected expense at a yearly rate of over $4 000 000 as applied to the gas stripping plants of Greater New York only, which figure may be increased to over $9 000 000 as applied to the whole country.

"6. If the above procedure is not followed, the loss of toluol production is estimated at 2 600 000 gal. per annum, which is sufficient for about 37 000 000 lb. of T.N.T.

"7. While it is extremely difficult to estimate the probable net conservation of gas oil which would result from the establishment of a 528 B.t.u. standard, it has been estimated that this figure for the entire country would probably not exceed 1 300 000 bbl. per annum as applied to the gas plants (including the Pacific Coast) not equipped to recover toluol.

"8. While it is understood that the continued use of gas oil at the stripping plants at the rate necessary to insure the full protection

of toluol expected, would not be prevented by the proposed order, the matter is so serious in its effect economically and otherwise on the Army Programme, that I have felt the problem should be presented to you in this way, realizing that it will receive careful consideration prior to final decision in regard to the issuance of such an order.

"Respectfully,

"(Sgd.) BENEDICT CROWELL,
"Acting Secretary of War."

APPENDIX IV.

"PROPOSED FORM OF REQUEST TO LOCAL AUTHORITIES REGARDING STANDARDS FOR HEATING VALUE OF GAS, BY THE FUEL ADMINISTRATOR.

"The United States Fuel Administration is undertaking to provide in every possible way for the conservation of fuel in order to minimize the shortage which is imminent under present war conditions and needs.

"The gas companies of the country are large users of oil, coal, and coke for making gas. After several months of investigation, it has been determined that a substantial reduction in the heating value of manufactured gas and the elimination of all illuminating power standards would, in many cases, result in a material saving of these fuels.

"In consequence of this the Fuel Administration has been contemplating the issuance of an order to all manufacturing gas companies in the country to reduce the heating value of gas delivered to consumers to 528 B.t.u. monthly average. Realizing, however, that such a reduction in the heating value of gas will, in some cases, reduce the cost of manufacture and also the quantity of service rendered and thus raise questions for local authorities to adjust, the Fuel Administration wishes to accomplish these desirable changes through the voluntary action of State Commissions or local authorities, rather than directly by its own order.

"You are, therefore, advised that it is the earnest wish of the United States Fuel Administration, as a measure essential to the national security and defense and the successful prosecution of the war, that you should, wherever conditions permit of the saving of fuel in this way, take such action as may be necessary to put into effect at the earliest possible date, not later than , a reduced war-time standard for manufactured gas as distributed to consumers, considering for this purpose a minimum of 528 B.t.u.

"The reason for suggesting 528 B.t.u. is that it is the lowest

that has thus far been tried out in practice in the United States. It is the minimum which has been in effect in Massachusetts since January 1st, 1918, and which has recently been adopted by the Connecticut Commission.

"It is the wish of the Fuel Administration that the gas companies should maintain as low a standard as is consistent with efficiency and good service, in order to conserve the maximum amount of fuel. Unless such a standard is put into general effect, there may not be enough fuel to supply the needs of gas companies generally throughout the country, and a large proportion, if not all of such companies, might have to be restricted in the use of fuel to an extent which would prevent them from maintaining existing standards.

"It is particularly requested that the regulatory bodies take such steps as will put this order into immediate effect, even in cases for which readjustments of the price for gas might appear proper to local authorities.

"Your compliance with these recommendations will be in the national interest and of great assistance and relief to the Fuel Administration and make the difficult task of rationing the fuel supply of gas companies unnecessary.

"The United States Fuel Administration does not wish under any circumstances, to restrict the fuel which toluol recovering gas companies require for the manufacture of gas and the recovery of toluol to the same extent as heretofore. Gas companies recovering toluol should, therefore, be permitted to operate in excess of this standard to whatever extent is necessary to produce toluol.

"Wherever the standards for gas quality are such as to restrict or prevent the satisfactory recovery of toluol, it is urged that they be modified when so requested by the War Department. Otherwise, this request as to a change of standard does not apply to any company producing toluol.

"As a further measure of conservation, the United States Fuel Administration urges local authorities to make every effort to obtain greater efficiency and economy in the domestic and industrial uses of gas. To secure a substantial improvement in efficiency and economy in the domestic and industrial uses of gas, it will be necessary to have the users of gas exercise care and intelligence in the choice of utensils and in the burning of gas. A large part of the gas made is used in cooking and frequently the net efficiency of such operations is not over twenty-five per cent.; by care it may be considerably increased. Gas companies can help the Administration by adjusting appliances properly and teaching their customers how to use gas economically. In the industries, also, care should be taken to use

gas but not to waste it. The use of mantle lights in place of open flame burners is to be encouraged, and pilot lamps should not be burned higher than necessary.

"The Fuel Administration will appreciate reply from the local authorities as to the action which is taken or contemplated on this request."

DISCUSSION

A. C. Klein,* Esq. (by letter).—In addition to the experience in reducing the candle-power of New York City gas, the Ordnance Department, in its operation of the Government toluol plants, was confronted with similar problems in the Cities of Minneapolis, Minn., and Detroit, Mich. Mr. Klein.

In Detroit, the Gas Company was required to deliver gas of 20 c-p. and 600 B.t.u. Late in 1917, representatives of the Ordnance Department requested the authorities to reduce the candle-power, so as to permit the recovery of toluol at two plants, as contemplated. The City Council adopted a resolution reducing the standard to 12 c-p. and 600 B.t.u. during the operation of the toluol plant. The heating value standard was later reduced to 570 B.t.u., and although these standards were maintained until after the armistice, no record of any complaints was brought to the attention of the Ordnance Department, which was in constant touch with the plant operation, and particularly on the lookout for dissatisfied gas consumers.

The gas standard in Minneapolis, prior to the war, was 16 c-p. and 600 B.t.u. After negotiations for the operation of a toluol plant had been completed, the City Council was requested to reduce the standard to 10 c-p., so as to permit the operation of the plant without the use of additional gas oil. The Commission acceded to this request, and, later, further reduced the standard to 10 c-p. and 570 B.t.u., at which it remained until the toluol plant was shut down by the Ordnance Department. As in New York City and Detroit, no complaints of poor gas service had come to the attention of the Government inspectors.

To the casual reader of Mr. Falk's paper, it would appear that the Ordnance Department occupied an anomalous position in requesting a reduction in gas standards on the one hand, and in opposing a reduction on the other. There was, however, nothing inconsistent in its attitude. Its only concern was to obtain the maximum quantity of toluol, so that the T. N. T. plants might be kept operating at 100% capacity. Therefore, whenever a lower gas standard was necessary to permit toluol to be extracted, the Ordnance Department advocated a lower standard; when a lower standard would result in curtailing the toluol production (as would have been the case had the 528 B.t.u. order been put in force), the Ordnance Department opposed such a reduction.

This narrative of the action of the Ordnance Department in bringing about a material reduction in candle-power of the gas delivered in several of our largest cities, and its failure to affect adversely the interests of the gas consumers of those cities will be of great interest to the gas industry.

* Philadelphia, Pa.

Mr. Klein.

For many years gas engineers have advocated that all candle-power standards be discarded, so that they might be enabled to take advantage of economical methods of gas manufacture and distribution, which they are unable to use when obliged to conform with high candle-power standards. Practically all regulatory bodies have recognized the soundness of this stand, and have substituted heating-value standards, setting them low enough to encourage the utilization of gas from by-product coke plants. However, our largest cities have been the ones to cling to the obsolete candle-power standard, claiming that a considerable number of gas users are dependent on open-flame lighting, and would suffer by a reduction in candle-power.

The experience in New York City, where the candle-power was dropped from 22 to 10, or 55%, without complaint from consumers, shows that there is no longer any good reason for retaining a candle-power standard. Standards of any kind are intended to serve the purpose of insuring to the purchaser a uniform quality of product, which quality should be as high as possible consistent with its price. In cases such as that of a public utility, where the price paid by the consumer is determined by representatives of the Commonwealth, the fair price necessarily bears a direct ratio to the cost of production, and public interest, therefore, demands that the standards should be such as to keep this cost of production as low as possible. The candle-power standard does not meet this test, and, therefore, it should be discarded.

In considering the question of gas standards, regulatory bodies should have broad enough vision to see the future sources of gas supply.

Our requirements for smokeless fuel are increasing at a very rapid rate, while the production of anthracite is increasing little or none. The shortage can be made up by the use of coke for domestic purposes. Coke is to-day a formidable competitor of anthracite, and the extension of its use will tend to keep anthracite prices down. The manufacture of coke in modern oven or retort plants yields a by-product gas which is in every way satisfactory for domestic cooking, heating, and lighting, although it cannot be manufactured to meet a high candle-power standard. To-day, more than a score of large cities, including St. Louis, Boston, Baltimore, Newark, Jersey City, Indianapolis, St. Paul, Hartford, Providence, Rochester, etc., are being supplied wholly or in part by such gas. None of the cities mentioned has a candle-power standard.

The authorities who are charged with the responsibility for protecting the interests of the public served by public utilities, and who have not yet recognized the wastefulness of the candle-power standard, should give full weight to the points brought out in Mr. Falk's paper, and should set standards of gas quality that will encourage the construction of large coal-gas plants. By so doing, they will be instrumental in creating a source of economical gas, as well as a source of smokeless fuel, which will act as a check upon the rapidly mounting prices of anthracite fuel.

Mr. Porter.

HORACE C. PORTER,* ESQ. (by letter).—The paper is particularly illuminating on the relationship of toluol production during the war to required standards for city gas. The author is to be congratulated on his clear and forceful presentation of the facts bearing on the treatment of this question by the War Department and the Fuel Administration during the war.

Attention may be called to an additional item, not mentioned by the author, in the increase of toluol production accomplished by the Ordnance Department. The coke ovens which at the beginning of the war produced 85% of the toluol supply, were assisted in obtaining coal of good quality, and in raising their yield of toluol per ton. An increase of more than 10% was thus obtained in their production, and four times that increase would have been possible with intensive effort and due co-operation on the part of all governmental agencies concerned.

The average production of toluol per ton of coal by coke-oven plants throughout the country was at the beginning of the war about 0.28 gal. During 1918, up to November 1st, ten plants (out of a total of thirty-five) averaged 0.44 gal. per ton of coal. The highest was 0.56 gal. With a coal mixture of reasonably high percentage of volatile matter (having regard for the necessary coke output) and with reasonably practical changes in operating conditions, the average yield of toluol from coke ovens throughout the country could have been increased to 0.40 gal. per ton, or by 40 per cent.

In December, 1917, the Fuel Administration was urgently requested by the Ordnance Department to take steps toward preventing the diversion of high-volatile coal from coke-oven plants, and to insure to these plants a full coal supply so as to maintain toluol and ammonia production. Belated steps were taken in this direction, but the results were not in evidence for several months.

The development of toluol supply from city gas plants was resorted to in the early days of the war because it would relieve the situation quickly. The old saying reminds us that "hindsight is better than foresight", but as we look back in the light of later experience it appears unquestionably true that the utilization of existing coke-oven capacity for toluol production at the highest possible efficiency and with fullest possible coal supply of highest percentage volatile matter would have yielded a large increase in toluol in a short time, and would have rendered unnecessary some of the more expensive of the Government installations at gas-works. Coke-oven plants under construction at the beginning of the war could also have been hurried to completion and their toluol production made available earlier.

Coke-oven plants and their benzol-toluol recovery equipment have a good post-war value, while city gas-stripping plants have little or

* West Conshohocken, Pa.

Mr. Porter.

none. Conservation is promoted by the former, and disturbed by the latter.

Had the Fuel Administration not failed to provide the coke ovens with the necessary coal for full toluol production in the early stages of the war, there would have been better ground for its subsequent plea for oil conservation at gas plants through the 528 B.t.u. standard and consequent toluol reduction.

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Paper No. 1446

THE NATIONAL RAILROAD QUESTION OF TO-DAY.

An Address Before the American Society of Civil Engineers February 5th, 1919.

By Francis Lee Stuart, M. Am. Soc. C. E.

With Discussion by Messrs. T. Kennard Thomson, A. J. County, George W. Kittredge, Calvin Tomkins, Augustus Smith, John H. Bernhard, G. J. Ray, Herbert C. Keith, Clement C. Williams, C. A. Morse, S. Whinery, and Francis Lee Stuart.

For an understanding of the Railroad Question of to-day it is necessary to review the facts which led up to Federal Control of the Railroads.

As one who had a hand in the development of several trunk line railroads with Ocean Terminals and has been otherwise connected with the Railroad subject in many ways, the important facts as I see them are:

For ten years before the war the clashing of State and Federal laws, the effect of the Laws under which the Interstate Commerce Committee worked or their interpretation of such laws gave indication that the railroads of the country, while in a high state of efficiency, were losing their attractiveness to investors, and many were losing their financial strength, so that the war found most of the railroads with small surplus in hand for meeting an emergency.

Early in our entrance into the war it became evident that the rush of material to the seaboard by all the unrelated efforts of the various departments of our Government and of the Allies, could not be handled by the shipping available, and that the physical and moral burden would fall on the railroads of the eastern section of the United States until

proper organization could be set up in the War Department to take care of the situation.

The eagerness with which every interest pressed the loading of cars, regardless of whether there was a prospect of prompt unloading of cars or not, and the inefficiency of the railroad embargoes began to congest the tracks, cars, and other facilities of the railroads, bringing with them their train of consequences, all of which tended to lessen the earning value of the properties; and, in addition, the buying power of the unit of value for which transportation service was paid rapidly began to depreciate, causing conditions which if prolonged would make a crash inevitable before the public could understand the real trouble and remedy it.

With our restrictive railroad policy, in order to save the transportation means of the country while it was still an efficient agent for winning the war, some Governmental aid had to be extended to tide over the situation. This aid finally took the shape of an Act of Congress, and the railroads were taken over under Federal control for the duration of the war and twenty-one months.

A Government agency equal in power to any other Government agency issued orders that cars were not to be loaded unless there was a definite arrangement for unloading; demurrage charges were increased and enforced; public opinion gave its moral support to orderly methods; the Government assumed the financial burden; and the railroads, as soon as the exceptional weather conditions of the first two months of Federal control permitted, with the same staff and the same equipment, began to function satisfactorily for war purposes.

A standard return was guaranteed for the use of the railroad plant; rates were raised so that the cost of transportation of the day should pay its way; wages were raised to meet the depreciating value of the dollar; and, in addition to those three vital essentials for handling the situation, the American practice of detouring and the placing of effective embargoes to avoid congestion was used on a large scale, and there was an insistence for a commercial business policy which provides a storage for unloading cars promptly to avoid the use of cars as a storage medium, all of which helped the transportation effort.

Under existing laws it was necessary for the railroads to be taken over for the winning of the war, and they were directed by the Director General of Railroads, for the President, with the single purpose of

making them efficient transportation agencies for winning such war, and the war is won.

The cost of a war must always be excessive, for obvious reasons, and there can be but little criticism of such actions as have been taken by this country, for when a republic is at war the resources of the nation should be utilized for the purpose of winning the war. Economic considerations, material and personal, must be disregarded when they interfere with that singleness of purpose.

It is best that the Governmental action of a republic at such a time should be as free as possible from criticism during the period of danger, and legitimate criticism can wait until time permits an analytical and impersonal investigation as a basis for criticism for the purpose of educating the republic in case of such a future emergency.

While on the subject of criticism, let me suggest that, although it may not be advisable to advocate that we draw a veil over all the acts of the war, yet we are all too prone to criticize, and let us in justice remember that the men who are responsible for getting our preparations under way and for the stupendous scale upon which we proposed to carry on the war, are the same kind of American Public School boys you and I are, and that we would probably not have done much better. And let us further console ourselves for our taxes with the thought that no one can tell what real effect the very scale of our effort had on the morale of the Germans, and how many thousands and tens of thousands of lives may have been saved thereby.

The war is won; the readjustment problems are serious, but, grave as they are, conditions are changed, and the happiness of the people and the due consideration of the economics of the country have become the paramount duty of the Government in solving these problems, and there is no reason why our re-adjustment problems should not be fully discussed.

Very fortunately for the country, an unusually able staff was selected for the United States Railroad Administration, but the fact remains that Federal Control of all the railroads as one road, with the best intentions and with bright minds giving their earnest thought to the Federal problems, while it has taught the country at large, and the owners, valuable lessons, which are worth all it has cost, has developed nothing of real material moment which it is desirable to perpetuate in peace times, or which has not been practiced under

private ownership, or cannot be more effectively practiced under private ownership.

The railroad question of to-day is what shall the Government do with the Railroad? Government ownership or Government control for five years, or a return of the railroads to private ownership as soon as possible?

Government Ownership.

Politics has no conscience, business morals, or responsibility, and the danger to our methods of government from the political control of the railroad employees, possible and probable, from Government Ownership, prohibits its serious consideration.

From an engineer's standpoint, this country is young, and its natural resources have hardly been scratched, and an experiment in Government Ownership would be extravagant, unfortunate, and undesirable on any basis, but primarily because it destroys the reward for initiative and sound judgment in the greatest field necessary for the proper development of our country and our national prosperity.

From the public's point of view, Government Ownership throttles competition, which is a necessary incentive for proper service to the people in every part of the country and every city in the land.

Control for Five Years.

Some of the things which were ordered with propriety during the war, which should be undone at once and not five years hence, are:

The Traffic Departments of the various railroads, which were dispersed, should in great part be re-assembled and re-habilitated at once. Such departments are as necessary in dealing with the public for the sale of transportation as the advertising and selling departments of any other business, and can be of great service to the public in many capacities.

An immediate abandonment as a national effort of an attempt at standardization of all manner of railroad necessities and methods. Such standardization, while it has a certain apparent first-cost economy when applied to quantity productions in the scheme of things in this country, tends to inefficiency, and should be a local question only in a limited locality. Our tools and methods are so crude that the national efforts should be to improve, not to standardize.

The horizontal raise in railroad wages of the entire country in one order which was necessary during the war is a pernicious destroyer of individual discipline, and atrophies the ambition of the men receiving such increase, and should be abolished in time of peace for responsible bargaining between employers and their own employees as soon as possible.

The discomfort of the public, some of which was necessary during the war, should give way to even better facilities than offered before the war, the direct tax of an increase of passenger rates should be reduced to a minimum, even if an indirect tax of increased freight rates is necessary to pay the bills. The movement of the people freely around and about our country should be encouraged as increasing happiness and social well being, and tending to a unity of spirit which is good for the body politic.

The discontinuance of regional consolidation, which was necessary for operation during the war, and an immediate return to a competitive basis in all matters pertaining to service to the public.

The terminal problem, which was made so acute by lack of ships and storage facilities, is largely one of changing our commercial methods, pooling facilities, and providing for the prompt unloading of cars. Belt railroads are needed and should be built in many places, but the matter is a local one peculiar to each community, and best handled as a financial policy of such community and not by the Federal Government.

We have great national efforts to make in world affairs, and the sooner we put the vim of an owner's interest back of the largest business interest in the country the better.

Return to Private Ownership as Soon as Enabling Legislation Can Be Passed.

This country is so young and undeveloped that the railroads should exist and be encouraged primarily for the development of our material welfare, and, for the good of all, the railroads should be returned to their owners as soon as possible so that their growth will naturally follow the normal business needs of the country tested by business standards and not directed into any pre-determined course which would be determined by a centralized Governmental agency.

Returning the roads to their owners under old conditions would bring financial disaster to insurance companies, savings banks, in fact, would mean financial chaos in the country; therefore, some legislation with certain Governmental controls is absolutely necessary.

The normal growth of our country requires that the railroads spend in the next five years at least four billion dollars for improvements, betterments, and normal growth of our transportation service, and a continuing and enlarging amount thereafter to meet our growing needs. While the Government finds it necessary to put out varying issues of Liberty Bonds, it seems best that they buy the securities of the Railroad as needed until the Public regains confidence enough to take the investment out of their hands. This will require a possible advance of from a billion and a half to two and a half billion dollars, for which the Railroads could be charged a high enough annual interest charge, over and above the cost of the money to the Government itself, to pay off the debt within twenty or twenty-five years.

It will take a stable continuing railroad financial policy to renew the confidence of the public in railroad securities, and it seems to me that the railroad interests of the country, which is the largest single interest, should write off all extraordinary war costs within a short generation and lesser business interests in less time—in fact, I think, to keep step with the age, the Government should find financial measures to write off and obliterate all or most of its national war costs within thirty to forty years or less.

The principle of a fixed financial policy and the control by the Government necessary to carry out such policy under all ordinary business cycles should be made a law by Congress. The control should be of an enabling nature and not more restrictive than is necessary, and might at the present time well consist of:

First.—A control of rates initiated by the carrier. The rate question is largely an artificial issue and little understood by the public. The shippers of the country are chiefly interested in rate relationships and the ultimate consumer in the amount of the rate, as he really pays the bill. We hear of the shippers through their lawyers, but the ultimate consumer is rarely heard, because his indirect tax due to the rate alone is so very small. There are possibilities in a more logical rate structure for settlement of most of the misunderstanding between

the public and the shipper, and between the shippers themselves, for which the railroads usually hold the bag. In connection with the control of rates, the Government should have an unbiased economic study made for a more logical application of rates than at present.

Second.—The wage question of the railroad employee, like other inflations of the war, will be settled sensibly by public opinion. It does not seem a proper Governmental function to settle wage scales, and it is for the best continuing interest of labor itself that such questions should be returned as soon as practicable to be settled between employers and employees. It is necessary, however, that the Government take cognizance of such matters in the control of rates.

Third.—A control of unproductive expenditures required by the public, and particularly as to the grade crossing elimination question which threatens to be of serious financial importance in the near future so that due cognizance may be taken of such expenditures.

Fourth.—A control of issue of securities for improvements and consolidations and of the necessary railroad accounts. Generally, the weak lines should be consolidated with the strong, and the large lines prevented from becoming so large that they eliminate competition from any section of the country.

Fifth.—A control in safety matters.

Sixth.—A recognition that for the continuing solvency of the properties the voice of the general public, whose welfare is at stake, and the voice of the owners of the properties, should be heard with that of the shippers and representatives of the employees.

In my opinion, the proper principles for legislation for a fixed financial policy should be based on an average railroad efficiently operated, receiving, as a minimum, a fair profit for the transportation it has to sell, and that fair profit must include:

1.—An adequate going return on the investment over the cost of proper service to the public;

2.—A return for such money as is borrowed for improvements required by the growth of the country, which will include the cost of the money plus an added equity to the corporation in compensation for extending their credit and brains as collateral for such money;

3.—The further right to build up such surplus as this class of business, wisely conducted, should have in reserve for emergency.

Nothing short of such a financial policy under private ownership will bring investors back to the support of the railroads and keep up the healthy growth of our country. Nothing short of such a financial policy, under private ownership, will give the people of our country a healthy sense of freedom and the right to demand competitive service for their happiness, comfort, and material benefit.

The matter of return of the railroads to private ownerships on a basis of such a financial policy can be applied at once without an absolute determination of results before the policy is adopted. For a willing trader, the securities and proper earnings on the investments of the average railroad with the many years of a repressive public policy putting increments of values into the properties, should not be difficult to determine within safe limits. On any given rate, some roads will fail and others will prosper, from many different causes, but the extremes can be regulated to meet the political feeling of the times by the beneficent use of an excess profit tax.

With the principles of a policy fixed, and controls whereby it may be made effective, the means for carrying out the purpose could be more flexible, and could consist of a number of commissions with some coordinating force, all safeguarded in certain cases by right of appeal to proper courts.

The entire matter is of such importance to the whole country that, if this session of Congress cannot settle the question, then an extra session should at once be called for this purpose. It seems to me, it is the simplest of our re-adjustment problems, and the one needing the most immediate attention, and the one most important for the immediate welfare of the country.

DISCUSSION

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—This is the most valuable paper the speaker has ever heard presented to this Society; its very brevity enhances its value.

The speaker is tempted to enter a lengthy discussion, but, for the sake of emphasis, will only mention two points, and those briefly.

Standard designs are all right in certain cases, that is, when the design is as nearly perfect as it can be made, and when used in the right place only. An old engineer once said: "A standard design is a stand-still design," and probably the most glaring offense in this line ever perpetrated on the public is the (can't sit still) design of the parlor car chair. The speaker has yet to find any one who was made to fit this uncomfortable seat.

As for Government ownership of railroads, the "proof of the pudding is in the eating thereof." It is known that the rates charged by the Government-owned railroads of Germany and elsewhere are much higher than in the United States, in spite of the shorter haul.

To come nearer home: Canada affords a very good comparison, for the Canadian Government spent $100 000 a mile to build the Transcontinental, and the Canadian Northern built an equally good road for $50 000 a mile.

Again: The Canadian Pacific Railroad spends $70 for interest and other charges to earn $100, and the Canadian Government spends from $200 to $220 for the same purpose.†

In other words, the Canadian Government spends twice as much to build and three times as much to operate as the privately owned railroads.

This is Government ownership as it was, is, and always will be.

Mr. County.

A. J. COUNTY,‡ ASSOC. AM. SOC. C. E.—The railroad situation has this gratifying feature: It demands thorough consideration, and, as the solution of the problem affects all citizens, the most experienced men should express their views freely. The speaker has listened with interest to many of the plans suggested to the Senate Committee in Washington, and congratulates Mr. Stuart on having "thrown into the ring" another plan which has so many sane features, and aims to preserve initiative and development in the transportation industry.

Should We Have Five More Years of Federal Control?—The situation impresses the speaker like this: The railroads have had Federal control and operation since January 1st, 1918, and, by Act of Congress, that control can continue for 21 months from the Declaration of Peace.

* New York City.

† Much valuable information about Canadian railroads may be found in a paper by W. F. Tye, M. Am. Soc. C. E., *Transactions*, Can. Soc. C. E., January–June, 1917.

‡ Philadelphia, Pa.

Mr. County. The country is requested to endorse 5 years more of such control and operation. That means uncertainty, with consequent disruption of the experienced managements and organization, and it would be politically unwise and economically unsound for us to continue in peace the form of operation and control that the war period necessitated. There appears to be nothing to prevent the return of the properties to their experienced managements and officers to operate the same for the Government during the 21 months that will follow the Declaration of Peace. That allows sufficient time to adjust the situation, permits the Government to fund loans made to the railroads, give them a final settlement of compensation due for the use of their properties, including deferred maintenance of roadbed and equipment, caused by lack of men and materials in the war period, and for Congress to adopt a constructive railroad policy for the future.

What Future Policy Should be Adopted?—As to such future relations between the railroads, the Government, and the public, we must have an open mind. Happily for us, the world has had experience on all plans suggested, so that we are not left to much guesswork or assumption in reaching a proper conclusion. There must be determined which one of the three chief suggested policies should be adopted as best for the welfare of the country and its interests here and abroad to produce the best and cheapest transportation service, viz:

1. Government ownership and operation, and financing.
2. A regional guaranty plan by which the individual railroads are to be amalgamated and operated by about a dozen regional non-competitive companies, and the present railroad securities exchanged on some basis for Governmental guaranteed securities of the regional companies, the Government to provide future capital.
3. Private operation, under strict Government regulation, by the existing railroad companies, with competition in service and effort to serve the public; the ownership of said companies to continue to be vested in our citizens and in their savings, industrial, and other institutions, and the citizens to provide future additional capital for railroad expansion.

Government Ownership and Operation Opposed.—We can at the moment say definitely that Government ownership and operation will not meet with public approval, although its advocates are by no means asleep. The experience of the world is against it, as a sound political and economic method. A nation may favor it for military purposes, or for some special national emergency or purpose, just as our own country assisted the construction of the railroads to the Pacific Coast and in Alaska, and many of our States constructed railroads and other public transportation facilities in the past, but it is opposed to our

forms of progressive democratic institutions as a whole, and the activities and temperament of our citizens. Mr. County.

Should Regional Railroads and a Government Guaranty Plan be Adopted?—Therefore, we must consider the second suggestion, that of one dozen Federal regional companies, owning and operating under Governmental guaranties. The argument for regional companies under a guaranty generally proceeds along these lines: Just as the Federal Reserve System mobilized and protected the resources of the banks, so the resources of the railroads can be mobilized and their operations unified. Such a statement ignores the fundamental framework of the Federal Reserve Law.

Federal Reserve Regulation Compared with Railroad Regulation.—It produced an admirable system of financial responsibility and cooperative effort, but do not forget that individual banks were left untrammeled (so long as they observed honest and business-like banking methods) to make as much or as little money as they chose for their owners and to strengthen their reserves. That left private ownership and initiative of each bank free to act within the limits of the law and good management. Consequently, the banks have been the bulwark of the country in the hour of its financial stress. No argument is used that the interest rates which allow the little bank to exist make the big bank too rich at the expense of industry or the public. We realize that the strong bank is better able to serve the Nation, and that its profits are ultimately regulated by taxation and the stress of financial depressions or losses, the same as railroad rates and profits are regulated. Banking, like railroading, affects every citizen and every industry, yet we are proud of our strong banks—why not of our strong railroads? The railroads have regulation that corresponds to the Federal Reserve Board and the Federal Reserve Banks, in the Interstate Commerce Commission and the State Commissions. The serious difference lies in this fact: the country enacted constructive banking laws, but punitive railroad laws. The Federal Reserve Board and the Federal Reserve Banks are required to be constituted so that they cannot become purely political institutions. The Boards are required to be directed by representatives of banking, commerce, agricultural, or other industrial pursuits, in addition to representatives of the Government, and, in the case of the Federal Reserve Banks, stockholders are also represented; but, in the State and Federal Commissions regulating the railroads, no such constructive provision has been made.

Further, in the railroad regulation we already have Regions, with the States as boundaries, for the State Commissions; and, for the Interstate Commerce Commission, rates are not dealt with for the entire country, but in the various traffic regions; and we have railroad systems, competitive in service and with the necessity to earn profits.

Mr. County. Therefore, theoretically, we have enough machinery for railroad regulation now existing, largely similar to that of the banks; but we need the mandate of constructive railroad laws; we need the concentration of responsibility for railroad results and credit; and we need to add to our railroad regulatory bodies, now exclusively political, experienced men representing the activities of the country in industry, commerce, and agriculture, men who should be strong enough to enforce a National policy to insure the development of the transportation facilities and the credit of the railroads. First give the railroads constructive laws, and commissions, representing the country at large, responsible to enforce those laws.

Further, the suggestion of non-competitive regional guaranteed railroads, though it might produce some more co-operation, wipes out the element of competition in service and in endeavor to serve the public. We already have competitive regional systems, like the New York Central and the Pennsylvania Systems, that include hundreds of smaller companies, and more can be done when there is an earning basis to justify it. These regional systems preserve initiative and develop the country, but the regional railroad plan would unify all the railroads in that region, and would have no such incentive. There is nothing in the railroad situation to prevent further unification of the various railroad systems and their feeders, provided proper legislation is passed. Take the Pennsylvania Railroad: There are still about 140 active companies in that system, but they represent what were originally more than 600 companies. There has been a gradual evolution toward big systems which are competitive in service and in management. That is the healthy thing for the country, but our restrictive State and Federal laws stopped the process.

Should we Have a Government Guaranty?—As to the Government guaranty: This regional plan and Government guaranty is practically the French plan. Under a Government guaranty there would be no inducement to take further risks. Here is a serious element against which the commercial interests of our country must guard or we will have stagnation. The speaker can do no better than refer to the testimony of the English economist and lawyer, Mr. Acworth, before the Senate Committee, in May, 1917, amplified by quotations from the eminent French economist, M. Leroy-Beaulieu, which will be added to this discussion. We need not go so far for results of the guaranty plan. In various parts of our own continent the regional guaranty plan has been tried. The Federal Government has tried it for 1918 with regional operations and a guaranteed standard return. As the railroads earned a good excess over that return under private operation, it was felt that the Government, with all restrictive laws and commissions eliminated, would earn substantial profits over its guaranty. The results for the three years ending June 30th, 1917, justified the stand-

ard compensation, which was equivalent to only about 5.2% on the road and equipment investment of the railroads. Just the same, whether we attribute it to war or other conditions in which economic results necessarily had to be overlooked, whether we say it is due to inefficiency or the great increase in the number of men and women employed, or the concentration of management from the local field to Washington, or the change from the trained managements, with private initiative and enterprise, to the regional plan, it produced a deficit, and we have not yet had the final figures. With all that, the speaker is willing to admit that the experiment justified the cost, considering the conditions confronting the country. The experience of Canada regarding the railroad guaranty will bear some careful reading. We know it is far from economic, and we know that the taxpayers will have to meet the huge loss. We have heard of the new era of "transportation for cost" on the Boston Rapid Transit lines, and the result will be an 8-cent or probably a 10-cent fare, or public taxation must meet the deficit. Indeed, we need not go outside of New York City to see what municipal ownership of the subways and the guaranty plan can produce. We know, fundamentally, that when a guaranty is given we take the first steps toward Government ownership, because the guarantor must assume the financial responsibility covered by its guaranty, and must also become finally responsible for the yearly capital improvements and financial budget, as well as for the final operating and maintenance results and service standards. Think of what it means in a country like ours, with changing political conditions and policies, and no party permanently responsible for the results. Mr. County.

Further, many of the advocates of the regional railroad guaranty plan require the practical financial reorganization of all the railroads in the country, the basis for which must be largely by capitalizing past net earnings or waiting for a final valuation, and, on the result, exchange existing bonds and stocks for those of the regional railroad companies. If any one can foresee the end of such a policy in a 10-year period, he is certainly optimistic. With present conditions, and with the load of war financing on the country, why add this additional load?

What Would Such a Government Guaranty Cost the Investor?—The railroad investor will first have to stand the results of such a financial reorganization and re-valuation of his property, and he must pay for the security of a Government guaranty. The general experience under the administration of such laws as the Railroad Valuation Law and the Federal Control Law is that they are slow and vexatious, and not by any means interpreted in a broad spirit, no matter what may have been the purpose of the framers. The investor, therefore, is bound to be sacrificed at the outset, and, in addition, under Federal guaranty

Mr. County.

and regional non-competitive operation, he, as a taxpayer, will have to share in the higher expense of conducting railroad operations when they are freed from the competition that exists under private initiative and management. He is held out the hope of a minimum guaranty and a share in the future profits. What Governmental body will allow the rates to exist to earn profits over the minimum guaranty? What commercial organizations would insist that rates should be increased for that purpose? The disturbing feature is that our advocates of Government ownership, or of Government guaranty, probably feel that it is no longer possible, under our laws, or through our Courts, to retain private initiative and ownership and yet secure a just return to railroad owners on their investment, so that there can be no safety to the investor except by putting the burden on the country at large. If we have reached that state with regard to a railroad investment of $20 000 000 000 that affects the entire country, we are no longer safe, even under a Government guaranty, in holding any railroad securities. Under such a state of mind, where will our country stand in getting its share of the world's business, and carrying it at the lowest rates? There are other suggestions as to guaranty, but the plan generally urged requires regional companies to unify the railroads regionally as described, believing that such non-competitive unifications will realize savings, and they will be disappointed.

Railroad Executives' Plan—Private Ownership and Operation.—To guard against the two extremes of this situation, viz: Government ownership and operation, and non-competitive regional Government guaranty companies, the Association of Railway Executives produced a plan, published prior to any other, based on their experience and continued study of several years, recommending continued public ownership by our citizens and their institutions, with private initiative and management, under strict but responsible Governmental regulation, with the ability to co-operate with each other, free from restrictive laws. It avoids political management, operation, and financing. That system, notwithstanding all criticisms, has in the past produced railroads giving the best and cheapest transportation, as a whole, of any system in the world, has the lowest capitalized cost, and has paid the highest wages and the highest taxes in the world.

Exclusive National Regulation.—Their plan calls for the exclusive National regulation of rates, of interstate commerce instrumentalities, railroad security issues, and important capital improvements. It provides for the separation of the judicial from the administrative functions of the Interstate Commerce Commission.

Interstate Commerce Commission a Judicial Body.—It constitutes the Commission as the judicial body to pass upon the reasonableness and adequacy of rates, with power to prevent all discriminations in rates or service, and to supervise railroad accounting and valuation.

Regional Commissions.—The Commission is assisted by regional commissioners, convenient to the public in their several States, with power, subject to the Interstate Commerce Commission, to deal with regional transportation questions and rates in their territory, and can recommend betterments of the transportation service and facilities. Mr. County.

State Commissions.—The State Commissions would continue their regulation of intrastate questions and corporations, which have become such a large feature in our daily life, such as trolley companies, electric light, gas, water, heat, power, and sewerage companies, and other features.

Administrative Department of Transportation.—The plan further recommends the creation of a Department of Transportation under a cabinet officer called the "Secretary of Transportation," so as to concentrate the responsibility on the National Administration for adequate transportation service and facilities for the country; to protect the just interest of the owners and employees, as well as of the shippers and the traveling public; to deal with administrative questions, such as traffic distribution, joint use of terminals, proper car supply, and other railroad administrative and executive functions, so that promptness and the responsibility of the National Government would be continually back of the railroads. This officer, however, would not have the powers of the present Director General. He would not operate the railroads, nor appoint any of the officers or employees, and therefore no unavoidable hurtful political influence could be exercised by him, any more than has been exercised in connection with the appointment, removals, and other features of the State and Interstate Commissions. Let us not delude ourselves that politics will be absent from railroad questions in our democracy. Let us, therefore, make it the most responsible political influence on which the eye of the public is continually directed, instead of scattering the responsibility for railroad results over 48 States and Commissions. The speaker has little doubt that if the public preferred a small National Railroad Board headed by the Secretary of Transportation, but including representatives of business pursuits like commerce, industry, agriculture, finance, or railroading, to deal forcefully with railroad administrative questions, like the Federal Reserve Board, there could be no reasonable objection, but the Secretary of Transportation must be responsible to the President and the country.

The Railroad Executives' plan provides for the funding by the Government of the loans to the railroads during the period of Federal control.

Railroad Consolidation and Co-operation.—It also recommends further amendments of the restrictive Federal and State laws to enable the railroads to co-operate with each other to provide joint facilities for public convenience, and to enable the carriers to acquire feeding and

Mr. County. connecting systems, and, to carry out more satisfactorily these beneficial forms, that Federal incorporation of the interstate carriers be permitted.

Rate Making.—The plan recommends that carriers have the power to initiate adequate rates, subject always to the approval of the Secretary of Transportation, and to final public review by the Interstate Commerce Commission as a judicial body, but rate initiation, suspension, and final disposition must be effected within 90 days, in lieu of the seemingly interminable periods in the past that so unsettled business and disrupted the railroads.

Statutory Rule to Fix Adequate Rates.—The final features of the Governmental control recommended by the Railroad Executives is a Federal law which shall include a statutory rule for making reasonable and adequate rates. If that power had been, or could have been, exercised in the past, there would have been no railroad problem, because the revenues would have been sufficient to offset rising costs and taxes, and attract the capital necessary to meet the railroad needs, and provide a basis to enable well established companies to sell stock as well as bonds on reasonable terms, and so prevented a great increase in the mortgage and fixed debt of the railroads. This rate-making rule has always been regarded as a difficult thing to lay down, because of changing conditions, and yet, in the decision of the Interstate Commerce Commission, they recognized that, when the situation arose in 1914 that the railroads earned only 5.36%, in the Eastern District, on their recorded investment in road and equipment provided for public use, the revenues were too low in the public interest. One can readily see that some minimum, like a return of 6 or 7%, on such property investment might be provided by statute, so that there would not be a repetition of restricted railroad credit, with its disastrous consequences and inadequate facilities for the commerce of a growing country. The speaker cannot see that such a minimum return would be objected to by the country at large, which has never desired weak or poorly maintained railroads, for that means business depression. This plan does not guarantee any return to any single railroad, but it will require, in substance, that the earnings of the railroads in the larger traffic regions of the country (such as the Official Classification Territory east of the Mississippi and north of the Potomac) should have an aggregate return which, as a whole, will always produce that minimum. This will still require each railroad to put forth its best efforts in operation and management to secure its share of the business and serve the public. The Railroad Executives' plan does not name any figure. Those the speaker has given are merely illustrations which, from his point of view, are essential to prevent future mistakes. It is quite usual for the State Commissions to fix rates of return permitted to public utilities. The water company serving the territory where the speaker lives is allowed to earn 7% in excess of operating expenses, taxes, and depre-

ciation. Whether or not the figure is imposed by statute, some protection is essential for railroad credit, because private ownership and initiative cannot continue unless returns are sufficient to pay operating expenses, taxes, and furnish a proper return on the investment so as to provide a proper basis for sound credit. No National railroad policy will be successful without embracing this rule of reason in rate making. That issue in railroad legislation and regulation can no longer be evaded. If it be conceded in legislation, and the railroads are liberated from the restrictive features of the anti-trust laws, and the State laws, what necessity is there for Government guaranties or Government ownership? Every guaranty or security issue of Governmental regional companies must weaken prices of other Government bonds, because the railroad guaranty concerns more than $19 000 000 000. Mr. County.

Wage Board.—The Railroad Executives' plan does not overlook the employees, nor the public necessity for uninterrupted transportation service. It provides for a Wage Regulatory Board, under the Secretary of Transportation, on which the public, the employee, and employers will be represented, so that the impairment of interruption of train movement, with appalling loss to the country, to the employees, and to the company, may be avoided, and lock-outs and strikes prevented.

Under such a far-reaching plan, the disturbance and economic waste heretofore caused by lack of a National railroad policy, and by divided responsibility, will be avoided, and there is no experimental feature about it. Like all other plans, it is, no doubt, susceptible of amendment, but it is hoped that our future National railroad policy will be adopted by Congress without Government ownership and operation, or Governmental guaranty of non-competitive regional companies, with consequent reorganization of the existing railroads, and with the resulting high transportation costs and paralysis caused by Governmental delays and restrictions, and inevitable stagnation to the future commercial progress of the country.

The following is the testimony of Mr. W. M. Acworth, of England, regarding Government relations to foreign countries, before the Joint Sub-Committee on Interstate and Foreign Commerce, before the Congress of the United States, in May, 1917:

The railway history of France stands by itself. Nowhere is the passion for logic and the love of symmetry which distinguish the French genius better exemplified than in the history of the French railway system. The ground plan of the policy which, broadly speaking, France has followed ever since, was laid down in a law passed as long ago as 1842. There were important developments after the world crisis in 1857, and again in 1878. And on January 1st, 1909, the Government, by taking over one of the six great systems, the Western, upset the symmetry of the original plan. But, with this exception, through-

Mr. County. out the whole history the original plan has stood firm. The whole country was divided up among six great companies, five of which radiate from Paris, and the sixth, the Midi, serves the extreme South and Southwest. The development of the railway network has been systematic from the outset; trunk lines first, then important branches, then the less important ones, and finally in recent years a considerable development of light secondary lines. Throughout, the State has guided, subsidized, and controlled. Each company has a monopoly of its own district. So far as possible, the points where the great systems meet are arranged not as in Holland, or formerly in Italy, at the great towns, but precisely at the points of least importance from a traffic standpoint. Where traffic is unavoidably competitive, as, for instance, from Paris to Central Switzerland, which can be reached either by the Eastern or by the Paris and Lyons Railway, arrangements are deliberately made to prevent competition. The Government controls all rates and fares charged and all services given, and the Government approves, not merely of pools, but of agreements by which shippers attempting to consign traffic by the route by which the railway companies have agreed the traffic shall not flow are deliberately penalized by higher rates. At a later stage of this paper more must be said of the work which the French railways have done, and reasons will be given to show that the French private companies have served their public at least as well as the Prussian State railways have served theirs. But it belongs here to deal with the history and the results of the nationalization of the Western Railway eight years ago.

For more than 30 years the French Government have owned and worked a system of lines in the Central West of France. This system was not inconsiderable in mileage (1 844 miles), but its district was purely agricultural and its traffic scant. It only reached Paris over the lines of the private companies, and, even when it served important towns such as Nantes and Bordeaux, it only served them by inferior routes. The financial results were very unsatisfactory—the operating ratio over a series of years ranged between 72 and 83%—but in some respects, as, for instance, admitting third-class passengers on all express trains, it gave to the public advantages which the private companies did not give.

Among the great companies, much the weakest financially was the Western. A large portion of its mileage was in Brittany, one of the poorest districts of the country. It did serve the town of Rouen and the very important port of Havre, but the canalization of the Seine diverted from the railway a large proportion of the valuable merchandise traffic between Havre and Paris. The French Government guarantees the dividend on the shares of all the French railway companies, and year after year the French Government had to find a considerable portion of the dividend to the Western shareholders.*

Further, large new capital expenditure was needed, especially to

* It should be noted that only 10% of the total French railway capital is in the form of shares; 90% is in the form of bonds which practically, though not formally, are guaranteed by the French Government. The dividend on the shares, therefore, though large in percentage—it ranges from 7% on the Western and the Eastern, and 10 or 11% on three of the other companies, to 13% on the Northern—is not very large in total amount. Broadly speaking, the companies other than the Western more or less earn their dividends in normal times without State help.

cope with the enormous passenger traffic of the company in the suburbs of Paris. How heavy that traffic is anybody who remembers the St. Lazare Station in Paris will know. The necessary capital could not be raised without Government approval; and Government approval, owing to strong parliamentary opposition, could not be obtained. Meanwhile, the service was going from bad to worse, and the equipment was on its last legs. Something had to be done. Once more the knot that could not be untied was cut; and, as from January 1st, 1909, the Government exercised its rights under the concession, and took over the railway, continuing to pay the guaranteed dividend to the shareholders in the form of an annuity running until the termination of the concession. Mr. County.

The political history of the transfer is distinctly interesting. The employees on the railway itself were unanimously in its favor; and, as subsequent experience proves, with good reason; and the whole strength of the Socialist Party and of socialist sentiment was thrown on the same side. A majority was secured for the bill in both houses, though a large majority of the representatives of the districts served by the railway voted against it. The bill passed the Chamber in December, 1906; but it was hung up in the Senate for 18 months, until June, 1908, and finally only passed by a majority of 3 votes, after M. Clemenceau, who was Prime Minister at the time, had given it to be understood that unless the bill was passed he would resign; and, much as the majority of the Senate disliked nationalization, at the moment they disliked the resignation of M. Clemenceau even more. It may further be added that, according to the French custom, the opinion of all the chambers of commerce in the country, which are statutory bodies, was invited on the measure, and that not one single town of more than third-rate importance reported in its favor.

Experience of State Operation in France.—Here in broad outline is the story of the result of the transfer.* In the 5 years, 1904-08, before the transfer, the gross receipts rose steadily from $37 084 000 to $42 145 000. In the 5 subsequent years, 1909-13 (the war upset entirely the figures for 1914), the gross receipts rose from $42 333 000 to $48 701 000, a slightly greater rate of increase. In the 5 years before the transfer, the operating expenses rose from $20 796 000 to $28 388 000, this last figure being unduly inflated by the fact that, from the date the chamber voted the acquisition of the railway, the staff became entirely demoralized. The first year after the acquisition the operating expenses were $30 304 000. The next year they were $34 921 000, and the year after, $39 454 000. In 1912 they had reached $41 800 000, and for 1913 they fell back to $41 478 000. In other words, to earn a net revenue increased by 31% the operating expenses increased by 100 per cent. The net revenue was never below $13 757 000 in the worst year of company rule. It fell to $5 352 000 after the Government had been in

* The figures given are from official sources. The story is mainly taken from an article by Pierre Leroy-Beaulieu, himself a Deputy, in "The State in Relation to Railways" (P. S. King and Company, London, 1912). See also Colson in "Revue Politique et Parlementaire" for November, 1910; December, 1911; May, 1913; and May, 1914. The railway newspapers all over the world were full of the story in 1909 and subsequent years. M. Colson, an engineer by training, was formerly head of the Railway Department of the Ministry of Public Works, and is now President of the Section of the Council of State (a body partly executive and partly judicial), which deals with all questions concerning transport by rail, or road, or water. His book, "Transports et Tarifs," is a recognized classic on its subject.

Mr. Bounty. possession 4 years. The operating ratio, which had risen from 56.4 to 67.8 under the company, was for the next 5 years as follows: 72.7, 79.9, 87, 89.4, and 85.2. It is fair here to make a qualification. The operating ratio had gone up on all the French railways between the two periods. It averaged 50.3 for the other five great companies for 1905 and 1906, and 58.4 for 1912 and 1913; but a rise from 50.3 to 58.4 is one thing; a rise from 56.4 to 89.4 is quite another.

A main argument used in Parliament in favor of transfer from the company to the State was that the company never did, and never could, earn its guaranteed dividend in full, still less the extra dividend which it would be entitled to if earned; that the company was accordingly a mere caretaker, and not interested in financial results, and did not operate, and could not be expected to operate, with economy. The State, on the other hand, would have a direct interest in operating economically. That was the argument. Here are the facts: During the last 10 years of company management, the State had to pay an average of $2 894 280 a year to meet its liability under the guaranty to make up the deficiency in net operating income. During the first 3 years after the transfer, the sums it had to find, under the same head, were $6 753 320, $8 875 792, and $14 934 484, respectively. For 1913, the figure was $14 752 237.

Nor do these deplorable financial results tell the whole story. The service to the public was absolutely demoralized. There were several very serious, and numerous smaller, accidents, and the staff and the public got so frightened that the express trains on the main line, already the slowest in France, were decelerated down to a timing that had been abandoned as inadequate in 1896. In addition, a number of trains were suppressed altogether. Punctuality went to the winds. Commuters on the system in the suburbs of Paris were compelled by their employers to live elsewhere because of the unpunctuality of their arrival at their work. As for the service in general, one figure will suffice. Compensation for accidents, loss and damage, averaged some $400 000 or $500 000 a year in the last days of the company. In 1911 the figure was $2 045 291. The Minister of Public Works himself publicly criticised the State administration as "a frightful fraud." And the Senate passed unanimously a resolution beginning: "The deplorable situation of the State system, the insecurity and irregularity of its workings * * *."

M. Leroy-Beaulieu gives the reasons for the "deplorable situation" as follows:

"In the first place, it is the abuse of formalism and red tape, with all the delays which follow and which are directly in conflict with commercial needs. In the second place, it is the lack of stability. The director and all the chiefs of the service change at the will of the ministers, whilst in the private companies the higher personnel is maintained a long time, fulfilling the same functions. It is next the political influence which enters into the choice and advancement of the personnel. It is, lastly, the lack of discipline which also results from the political influence at work. From the electoral point of view, the lower staff, being much more numerous, will always have much more power than the superior staff. It is always on the side of the former that many deputies will be systematically ranged. Above all, it is impossible to be at once controller and controlled. If one of the great French

companies under private management renders poor service, the public opinion is not slow to move the public power, and as this has the means to bring pressure indirectly but in many ways upon the companies, they are led to reform. On the contrary, when complaints are made against the State itself, the administration, irresponsible, does not listen. Rather, it seems indignant that particular individuals or even large associations should dare to find that all is not perfect. 'I have not seen without a certain astonishment,' said the minister of public works, in the chamber, 'the chambers of commerce criticise the actions of the State in its (their) reports upon the railways.' The same minister has dismissed an employee from his office who was at the head of a section of a passenger line which emitted protestations against the delays on the western State service in the Paris suburbs. When the minister was reproached in the Senate, he declared himself ready to do the same again. One may see how dangerous to the liberty of citizens the extension of the industrial régime of the State would be, where the number of functionaries would be indefinitely multiplied and where they would no longer have the right to complain. Mr. County.

"From all points of view, the experience of State railways in France is unfavorable, as was foreseen by all those who had reflected upon the bad results given by the other industrial undertakings of the State, such as telephones, matches, and many others. The State, above all an elective administration, cannot be a good commercial manager. It works expensively, and is powerless before its employees. The experience which we have recently gained has had at least one result. It has provoked a very lively movement, not only against the repurchase of the railways, but against all extension of State industry. This result seems to me fortunate. I hope this opinion will be maintained and that not only we, but our neighbors, may profit by the lesson of these facts."

M. Leroy-Beaulieu gives examples of what has happened under the various heads. "As for formalism and red tape," on the eve of the handing over of the railway to the State there were 1 526 employees in the central office. Within three years the number had increased to 2 587. "The single service of the accountant general was increased by 70 persons directly after the repurchase." And this was partly due to political pressure and partly to excessive red tape. For example, in the Caen Division, the preparation of the pay sheets which, under the company, took 9 persons, 3 days = 27 days, under the State administration took 12 persons 6 days = 72 days. "According to official documents, there are not less than 96 persons receiving a salary of more than $1 929.52 in the State system" as against 33 on the system of a neighboring company of much the same mileage, but with much higher receipts. In the 5 years from 1908 to 1912 the total expenditure increased $10 573 770, of which $8 412 707 were for salaries and wages.* In 1908, out of every $19.30 of receipts, the company paid $7.24 in salaries and wages. In 1912 this figure was $9.70. The comparative figure for the five great companies is only $5.70. In 1910, per $192 952 of receipts, the State railways employed 235 persons, the private companies 174.

* The 1912 figures are budget estimates only.

Mr. County.

Again, as the result of the great strike of October, 1910, the Chamber of Deputies voted a large all-around increase in the wages of the staff. Naturally, the staff thought there were shorter cuts to increases of wages than hard work. As a sample of want of ordinary business management on the financial side, the budget commission of the chamber reported that it could not draw up a proper estimate for 1912 from lack of the necessary accounts. "The statements addressed to the budget commission by the administration were manifestly inaccurate. * * * The great part of the statements of receipts and expenditure were found to lack any sufficient justification. Thus, as regards the expenditure upon personnel, the tables accompanying the statement only stated the numbers of workmen and staff and the amount of their salaries in round figures. * * * We wrote for further statements, more particularly the numbers of the personnel. * * * The minister replied (three years after the railway had been taken over) that the enumeration of employees had not yet been made. * * * This example suffices to show the trust that may be placed in the other parts of the budget estimate." This is not the opinion of an outside and unfavorable critic. It is the report of a committee of the chamber of the same political complexion as its predecessor, which voted for the purchase, based upon a draft drawn up by a deputy well known as an advocate of State ownership.

One story given by M. Leroy-Beaulieu is so striking, as a sample of the highest quality of red tape, that it deserves to be given at length. It is from a letter of a station master read in the Chamber of Deputies:

"In the time of the Western Company we station masters had orders to use the rolling stock as quickly as possible and to send to a given station all that we did not ourselves require. Under the State all is changed. Every station master is forbidden to load any wagon without the orders of the distribution bureau of the district. This bureau is, as is well known, a new creation, specially designed for the purpose of finding situations for so many more bureaucrats. Recently, having received two wagons loaded with horses, accompanied by an order to send these wagons to Caen after they were unloaded, I thought to do well by loading in these two wagons 200 sacks of grain, which had been waiting in the sheds for several days to go to Caen. But, alas, I did not know the bureau of distribution. The next day I saw my two wagons return, and I received at the same time an order to unload them. I was reproved into the bargain for excess of zeal. I had to obey the order. That evening I sent the wagons empty to Caen. Next day I received two others, also empty, in which to load the grain."

Mr. ttredge.

GEORGE W. KITTREDGE,* M. AM. SOC. C. E.—The Society is to be congratulated on having Mr. Stuart speak on the railroad situation, for no other member is better qualified. He has had to do with the construction and operation of railroads, has been connected with the Federal Administration, and is qualified to judge of their requirements and to realize their position.

In reading this paper, before coming to the meeting, the speaker was reminded of the story of the countryman who had been reading

* New York City.

Plato, and said: "I rather like that man Plato; he has got so many of my ideas." Mr. Kittredge

Mention has been made of the fact that railroads seldom hear from consumers in regard to rates. There may be good reason for this. A rate that is of material benefit to a carrier may have an almost unappreciable effect on the actual price of commodities. For example, an increase in freight rates was asked by the carriers a few years ago, which, if fully granted, would have substantially increased their revenues, and have given them, at that time, a living income. This increase in rates, which meant so much to the carriers, made a difference in the cost of a pair of shoes transported from Lynn, Mass., to Chicago, of one-quarter of a cent per pair.

Emphasis has been placed on the necessity of something being done to take care of unproductive expenditures, such as those in connection with large terminals, and with grade crossing elimination. With a large terminal, like the New York Central, if it were not possible to secure revenue from tenants in, around, and over the station and tracks, the carrier would be very severely affected financially by the enormous expenditures.

Where grade crossings are eliminated, the cost often runs into tens and hundreds of thousands of dollars, and yet it is doubtful if one dollar of added income comes to the carrier by reason of the grade separation.

It costs more to handle a commuter from Yonkers or Mount Vernon through the Grand Central Terminal than the gross amount of money received from that commuter for his ticket, leaving to the carrier the privilege of bearing the expense of housing him at his outlying station and conveying him to and from the city.

On one of the railroads terminating in New York City there are projects being urged for terminal improvements, grade crossing eliminations, and the like, which, if carried out, will involve expenditures of $25 000 000. If some equitable and comprehensive scheme could be devised for taking care of such expenditures as would be unproductive of revenue, those projects might be carried out with benefit, safety, and comfort to the public, giving employment to men returning from military and naval service and war activities, and largely benefiting communities in which such projects are to be undertaken. It is out of the question under present conditions for the carriers to consider such projects. Facts of this sort are not realized as they should be by the public, and Mr. Stuart has done well to dwell on the necessity of making provision in some way for such unproductive expenditures. Such expenditures must be made. They are necessary, from the standpoint of the carrier, as well as from that of the public, and if criticism is to be offered in regard to the amount to be thus expended, it must be that too little, rather than too much, has been expended in this way

Mr. Kittredge.

in the past. Those deeply interested in the railroad situation would do well to give this phase of the subject careful attention.

The welfare of the entire country is intimately affected by the railroad situation. The producer, the shipper, the merchant, the consumer, the investor, and those who prudently try to provide for their families and dependents after their own working days are over; all these, added to the hosts of employees who are necessary in the construction, maintenance, and operation of the carriers, make a vast army of people whose welfare, directly or indirectly, depends on the stability and financial integrity of the railroads. Statements, made in moderation, giving facts and statistics, together with sane discussions such as presented in this paper, are needed and should be encouraged; for the value and necessity of the railroads, and the well-being of the country, are too great to be jeopardized.

Mr. Tomkins.

CALVIN TOMKINS,* ASSOC. AM. SOC. C. E.—The speaker has not had an opportunity to read this paper and cannot discuss it as carefully as he would like. It is fortunate that a gentleman who knows so much about railroads, and who has also had Government experience, has undertaken to discuss the question at this time.

There is no doubt that competition under private initiative, such as we had during the period when our railroads were being developed, is more economical and more efficient in every way than Government ownership and operation; but have we sufficiently considered changed and changing conditions? If there is any way by which the country can preserve the advantages of private initiative, by all means it should do so.

The railroads of the country were developed under the leadership of its brightest minds, by a process of the most intense competition. That competition, consciously or unconsciously, had one end in view, which was the consolidation of the many railroad systems into one. For fifty years steady and obvious progress was made away from many systems toward a few systems, and an unconscious progress toward one system. We have attained the ultimate end of this process of natural selection, and the only reason that the process did not reach its logical conclusion under private control, was because the War intervened and precipitated the end under Government auspices.

The Sherman Law temporarily interfered with the personal consolidation of systems, and prospective bankruptcy stared most of the roads in the face as a consequence of the National policy of starvation which increased expenses without permitting corresponding increase of revenue. The railroads were not permitted to raise their rates, and their living costs kept on increasing. This greatly accelerated the tendency toward failure and ultimate absorption of the weaker by the

* New York City.

stronger roads. The War, however, brought out the defects of the system under the Sherman Law, and as these defects became apparent, the separate systems were put under one control for war purposes. Mr. Tomkins.

From April until September, 1918, the speaker served as a member of the Inland Waterways Committee, a branch of the Federal Railroad Administration. The Committee made its report as to what should be done with the waterways to help out the railroads, which in the Spring of that year could not carry the freight. Executives were put in charge of the Mississippi River and Erie Canal, and traffic on them was developed.

Not later than September in that year the situation had entirely changed. The consolidation of the railroads and the efficiency of their operation as one system under the best executives, had brought about such a degree of relief that the aid of the waterways was no longer needed. Freight was even diverted from the Erie Canal to the railroads, and there was much criticism in New York State for permitting this and thus limiting the use of the Canal. The principal freight carried on the Canal was grain from Buffalo eastward; and Mr. McAdoo, at that time, acting as the Trustee for the owners of the roads, was protecting them, and properly doing so, against the competition of the waterways.

Mr. McAdoo has asked, since he left office: "What is the use of developing the waterways system of the country if it is to be put back into the old competitive relation with the railroads?" The railroads had virtually succeeded in destroying water competition, except coastwise and on the Great Lakes, and this was necessarily done as a part of the competitive game in which they were engaged.

The situation will in truth be a difficult one for the railroads and for the canals if the old destructive competition is permitted to reassert itself, and the speaker will now ask the complementary question: "What will become of the railroads under such circumstances?" Waterway competition has immense possibilities, which thus far have not been realized. Heretofore, the waterways, and especially the Erie Canal, have been mismanaged by politicians, unprovided with adequate terminals or any reasonable system of prorating with the railroads.

Now that the public is interested in the waterways and highways, there will arise an insistent and increasing demand that there shall be established a fair system of prorating between rail and water commerce, and that the waterways, like the highways, shall be modernized in all respects. If the railroads shall again temporarily go under private control, this waterway competition will without doubt be intensified. The Railroad Administration, which is still very properly acting as Trustee for the holders of railroad securities, has held in check this competition, but if the Government shall in the future encourage full and free competition between quasi private railways, and public water-

Mr. Tomkins.

ways and highways, the result cannot be otherwise than disastrous for the railroads and, the speaker may add, harmful to all three factors.

Any one who has witnessed the volume of traffic on the Rhine and its connecting canals knows the possibilities of waterways, without many locks, such as we have in America and such as exist in Germany. In the speaker's judgment, Government control over these three factors of transportation is essential, if we are to avoid the uneconomic consequences of unnecessary competition. The railroads have also to consider the question of competition with modern highways. A large part of their most profitable traffic has heretofore been the short hauls; but now, any haul over good roads within 50 miles from point of origin to point of destination can be performed more effectively by motor truck than by the railroads handicapped by the intervening transfers.

Existing public sentiment may perhaps be temporarily against Government control, due to recent wartime experiences, but the speaker believes the pendulum will soon swing the other way, since, judging the future by what has occurred, we perceive that a steady progress toward consolidation has characterized the railroad history of our country, which has been a record of exploitations, absorptions, expansions, and bankruptcies, and then the same processes repeated over and over again, the units being larger each time. That is what occurred under the most efficient railroad generalship that has ever been had. It has been the lack of guaranties, lack of any certainty or safety, and the great risks incurred, which in the past made the railroad systems of the United States so effective. All this was accomplished under a régime of most intensive competition. Projects to raise fur on domestic animals have never been successful. It requires the hardship of exposure and wild life to bring out the fur. It is the same way with railroading. It required the law of natural selection, operating freely, to bring to the front the kind of men who developed our railroads, and produced such wonderful results, culminating at last in the greatest railroad system of the world. The competitive conditions which produced these men and these grand results, however, no longer exist. How, then, is it possible to expect to base any system of railroad operation on them? How can we provide competition, when the elements of competition are no longer here?

Are not the proposed "regional systems", equipped with "Regional Directors", in supposed "service competition" with each other, merely camouflaged semblances of the older order of real things which has passed away?

Is not this proposed machinery a mere pretense which may obstruct rather than serve the purposes of operating a national railroad system, already unified?

Are not the proposed regional zones, with their directors, comparable to the several floors in a single department store with their respective floor managers in charge?

Mr. Tomkins.

The Government determines freight rates, wages, hours, schedules, routing, joint operation of terminals and ticket offices, safety devices, issue of securities, uniform accounting and, most important of all, credit—and there is general agreement that these Governmental powers should be perpetuated. What else, aside from the purchasing and allocation of supplies, is there which can possibly be delegated to private initiative? The genius of Commodore Vanderbilt, C. P. Huntington, E. H. Harriman, and J. J. Hill was not exercised under conditions such as those just outlined. The ultimate object for which they, consciously or unconsciously, competed so strenuously, and which has at last been achieved all the more quickly as a result of the War, was a unified system, and railroad competition is now as dead as they are.

A Government guaranty, direct or indirect, to the effect that rates shall be sufficiently high, both to recompense invested capital and to attract new development capital, is, in the speaker's judgment, and under existing conditions, less desirable than will be a public valuation of railroad securities, accompanied with a guaranty of interest based on such valuation. All rate making and responsibility for all expenditures should be left to the general Government which, in the end, must hold the bag, under all suggestions which have so far been made.

A guaranty of payment of interest on outstanding securities is a simple and direct obligation of the Government, easily understood, and, if once undertaken, not likely to be avoided. The delegation of the power to initiate rate making on a basis sufficiently remunerative to provide for all contingencies, and to include a speculative profit in addition, will be a more complicated obligation for the Government to enter into, and one which is much more likely to be avoided in the future. Speculative profits are elusive. Interest payments on railroad securities transmuted into Government bonds are definite obligations.

As to a division of prospective profits between operators and security owners to serve as an incentive to enterprise, the answer is that it is not likely that there will be profits, and, if there are, the operatives will demand them all and probably will get them, to the exclusion of the holders of securities.

The unscientific existing railroad rate structure must be revised. It underlies many of our past railroad troubles, and is the resultant of a variety of forces operating over a long period of time, prominent among which are to be noted the following:

Actual or potential water competition; "what the traffic will bear"; "what was necessary to move traffic", and, finally, equalization of the opportunities and defects of localities.

A juster appreciation of the cost of service and the separation of terminal from line haulage charges must be given greater consideration in rate making hereafter, or the annual railroad deficit may be disastrous. Interstate Commerce Commissioner Robert W. Nolley has ably discussed this matter.

Mr. Tomkins.

The average security owner is as careless of his vote as is the average citizen, and, consequently, ownership, whether public or private, is comparatively immaterial. The roads have always been run, are being run, and will continue to be run, by expert operatives, from the track boss up through the engineering and accounting staffs to the President and the Director General, and the sooner this fact is recognized by the Government and the public and by security owners, the better it will be for all concerned. The quality now most needed for efficient operation is the development of *esprit de corps* in the operating staff, which in time will generate efficiency—not indeed as satisfactory as that which existed before competition ended, nor as bad as politics alone can otherwise make it.

Efficient services rendered to the public, not for bonuses and tips, but for just wages and salaries, in a distinguished and honorable employment, is what the country requires and may ultimately expect from the operating staff.

Absolute security founded on a just basis of values is what the security owner most needs—in other words, a Government bond—and if he delays much longer to press for this, in the hope of securing the elusive chance of higher speculative profits, based on politically delegated control over rate making, he is likely to miss his market.

Mr. Smith.

AUGUSTUS SMITH,* M. AM. SOC. C. E.—The speaker is not a railroad man, and will not attempt to discuss this paper from a railroad standpoint. As a freight shipper, and as a typically insignificant "private owner," if that is what a stockholder or bondholder is called, he does not think that the railroads as a whole have been particularly well run when under so-called "private ownership."

For instance, though the Baltimore and Ohio Railroad has been improving steadily in the last ten or fifteen years (and doubtless a large share of the credit for this must be due to the author), for many years before that, the communities served by it were afflicted by a poverty stricken railroad, and the blighting effects of that condition can be seen to-day in Staten Island.

The list of badly financed, or exploited, and wrecked, railroads in the United States is a long one, and the list of people who have lost money in railroad investments includes nearly every one who has laid aside money since railroads were first started. The New Haven crash will not be forgotten soon in New York or New England. War or Legislative interference had nothing to do with this. It was one of the most "privately owned" of "privately owned" railroads, and seemed to have enjoyed the acme of private management.

The concluding paragraph of the paper states the following fact that every one will endorse:

* Bayonne, N. J.

"The entire matter is of such importance to the whole country that, if this session of Congress cannot settle the question, then an extra session should at once be called for this purpose." Mr. Smith.

We are now faced by the problem of reviving business and sustaining wages. The railroads, in time of peace, are the greatest purchasers of materials and building construction, and hence are the greatest agencies for indirectly employing labor, in addition to their own enormous pay-roll. Their ability to purchase and thus develop their property was curtailed for several years before the war by stringent legislation and by alienated public confidence. The railroads to-day, therefore, present the greatest opportunity for active constructive industry to furnish employment and maintain wages, if they can be promptly put in a position to catch up with their retarded development.

Furthermore, general industry is so dependent on railroad service and railroad rates that any uncertainty in the railroad situation is bound to act detrimentally on the general resumption and expansion of activities.

The author points out the political dangers and the lethargy and stagnation that would probably come with Government ownership, and evidently thinks that "private ownership" with proper regulation and control is to be preferred, notwithstanding the experience we had before the War; but, were it not for the loyal railroad engineers, the "owner" or "owners" of a railroad surrounded by labor unions, Interstate Commerce Commissions, State Legislatures, grade crossing laws, damage suits, lawyers and juries, and represented by financiers, would be in a sorry predicament.

The Federal Government has made great expenditures, and contemplates making much greater ones for the improvement of our inland waterways. It is considering the development of an Atlantic coastal waterway. It has taken over the Erie Canal and the Cape Cod Canal. It has long been in charge of navigation on all tidal rivers and estuaries and on the Mississippi River and the Great Lakes. None of these waterways, except the Great Lakes, can be appreciably utilized for transportation of freight, except in conjunction and co-operation with the railroads. The railroads, hitherto under private management, have deemed it to their own best interests to deny co-ordination with any water-carrying artery, and when it was impossible to throttle and paralyze the movement of water-borne freight, they have deemed it necessary to enter as savage competitors on the water.

The country is now launched on a great shipbuilding programme, and we are wondering how we can get the products of our country and its overseas purchases to and from the ocean and interior in the most economical and expeditious way, quite irrespective of giving the haul and greatest freight return to this or that railroad corporation. We are wondering how our ports should be arranged to reduce the tariff or tax

Mr. Smith.

exacted in the way of transfer expenses at the point of transhipment from rails to the sea.

We have watched for years the growth of interlocking directorates and money interests, whereby the same individuals and corporations are monetarily interested in the railways and in great industries depending on them for an outlet, and whereby, in various ways, such individuals and corporations have obtained an unfair advantage over competitors in special railroad rates and railroad service, and we have tried to remedy this evil by legislation and other makeshifts.

We have long been accustomed to itineraries of carload shipments that remind one of the travels of "The Flying Dutchman". L. C. L. shipments passing over several lines to their destination need not be expected until they turn up of their own accord. In shipping from Bayonne to Boston, for instance, the railroad is not comparable with trucking to New York and then by the steamship to Boston, even at considerable greater expense by the latter route.

We probably are all willing to concede that Federal management of the railroads would mean freedom, or at least equality of land movement—freedom of the land—and would permanently abate the preferential evils we have been trying to reach by legislation.

We may be able to visualize partly the great economic advantages that would ensue if the water routes and the railroads were operated so as to feed each other with business, and if the ports were operated as links in our transportation system, having a duty to perform to the producers and consumers, unmixed with the perquisites of a quasi custom house. Great improvements in handling freight are possible. The rates for carload shipments can probably be reduced. The time for all shipments can be greatly reduced. The suspicion or certainty that some interests get an unfair advantage over competitors in railway service can be done away with entirely.

The railroads are our long-distance highways—turnpikes of the first class. There is no more reason for their being operated by and for the benefit of private individuals than that we should return to the old system of privately controlled turnpikes. Of course, the present owners of these highways must be equitably compensated for their property, and the provisions of the existing contract whereby the Federal Government took over the railways, if the properties are to be retained by the Government, are probably satisfactory to the great majority of holders of railroad securities.

Let us keep before us the conception of a great system of highways owned and operated by the public. We do not need to include every railway in that system, nor do we need to include the operation on such a system of highways of all vehicles by the public. Pullman service, dining cars, refrigerator cars, coal cars, *trains de luxe* for passenger

service, long-distance freight, and all express service could be properly operated over the public highways by private enterprise. In fact, the railroads have been doing this very thing for years with each other, over leased lines, and by leased privileges. All that has to be done now is to let the Government hold the lines, or such of them as seem necessary to the Federal management, and be the landlord. The old railroad men will be available to provide car service. If new lines have to be built, private capital and initiative can be enlisted on equitable terms, and if politicians or self-seeking communities try to have new railroads built at Federal expense, the economic advisability of the proposed construction could be passed upon by Federal engineers, who would be as able and incorruptible as have been the U. S. Army engineers for years past in dealing with our navigable rivers and harbors, holding down expenditures to what would actually be of some benefit to commerce. Mr. Smith.

One of the evils that has impoverished railroads, and that the general public is now going to pay for in the increased transportation charges, is the abuse of grade crossing legislation, mentioned by the author. Baiting railroads has been popular in every State Legislature and town council. If the roadbeds belonged to the Government, grade crossings would be eliminated where necessary, but this would cease to be a political and popular sport.

It might be feasible to have the Government take over with the roadbeds and terminals—that is, with the real estate—the locomotives, train crews, and signal or communication systems. This principle has long been proved practicable under private ownership in the many instances where the trains of one railroad corporation are hauled over the roadbed and into the terminals by the locomotives and train crews of another. Under such an arrangement, strikes threatening interruption of mail service and even starvation for cities could not occur.

If competition is necessary for progress, as the author says it is, it would be possible to have several different corporations competing for business on the same rails, just as the Pennsylvania Railroad and the Central Railroad of New Jersey have competed on the right of way of the New York and Long Branch Railroad for many years. In fact, if the Government chose to put on a car service too, to make a base rate and provide, for argument's sake, a base service, we might rely entirely on competition to regulate the rates charged by private corporations, and dispense with the Interstate Commerce Commission, or at any rate, relieve that Commission of many of the duties it had before the War.

If the Government handled the locomotives and train crews, it would be responsible for train accidents, and some sort of accident compensation, fixed by schedule, would take the place of damage suits, juries, lawyers, and punitive verdicts bearing no relation to the money paid to the railroad by the victims of the accident for their tickets.

Mr. Smith.

Finally, the railroads on which people depend for service when they settle in a country and invest their life and money in some enterprise, would be taken out of the stock markets and placed under a guardianship that could not go bankrupt or be re-organized, and that would treat every citizen in every part of the country by the same policy.

Private ownership is a thing of the past. It has long since been a name only. Public ownership exists now. How shall it best be administered? Shall we go back to the fictions and elaborate artificialities of the recent years, made more elaborate by statute upon statute, as one difficulty succeeds another, just as any lie rolls along, growing like a snowball; or, shall we recognize the true relation between control and ownership?

Mr. Bernhard.

JOHN H. BERNHARD,* ASSOC. M. AM. SOC. C. E.—Undoubtedly, the author has advanced very valuable ideas on the question of public or private ownership of railroads, and yet it seems to the speaker that the seat of the trouble is deeper than his suggested remedies go. The underlying difficulty in the railroad problem is the rate system.

Using the term in its broadest sense—that of determining the amount to be paid for service rendered—it is evident that, if a satisfactory rate system is adopted, it matters little whether we have public ownership or go back to private ownership; in fact, it seems logical to assume that, with such a satisfactory solution, even the most ardent supporters of public ownership would become lukewarm, unless they were advocating such public ownership for political spoils, and thus disqualifying themselves.

The objection to the private interest is not against men, but against their methods and systems. The public would not be better off with Government ownership under the old system.

The rate problem can be summed up in a very few words: the so-called rate system is no system. It is an impossible fabric of the following principal considerations:

What the traffic can bear;
Anything to beat water competition;
Anything to beat competing lines;
Anything to secure tonnage;
Compromises—increases or decreases in rate based on what some other community has to pay to reach similar destinations, etc., etc.

We have no rate system. It just so happens: that is all that can be said for it. It grew naturally. It is self-made in its worst results. To a great degree—more so in the past than at present—the railroads have, if anything, helped increase its entanglements and its difficulties. Although the United States grew in a most remarkable way, the rail-

* Amsterdam, Holland.

rate system just grew locally, and, later, was patched together, with the result that this rate scheme is based on everything except cost-plus-profit. Mr. Bernhard.

A few illustrations of freight rates may elucidate this: Before the war, in carrying salt from Syracuse to St. Louis, a direct distance of about 900 miles, it was shipped *via* Baltimore, a distance of about 300 miles, reloaded at a cost certainly not less than 30 cents per ton, carried by water to New Orleans, a distance of 1 700 miles, reloaded again at New Orleans, again at a cost of not less than 30 cents per ton, and then carried up by rail to St. Louis, a distance of 700 miles, cheaper than if carried direct to St. Louis. It must be clear that the people as a whole pay somehow for this loss of labor, time, and misuse of investment; and the statement that such a commodity thus carried around may be moved cheaper than in another country is begging the question. The point is that it is moved 1 000 miles by rail and 1 700 miles by water, and handled twice, whereas it could have been moved 900 miles direct, without such rehandling. We should look on foreign countries as our rivals—friendly, perhaps, but rivals nevertheless—and, therefore, we should practice economy as far as we can, and not content ourselves with a statement that as a whole our freights are cheaper than elsewhere. Let us take advantage of our natural resources and not waste them, even if, in the minds of some, we have sufficient margin of profits.

If flour, a difficult article to carry, with its inevitable claims for damages, is carried from New Orleans to St. Louis at a rate per ton-mile which is equal to or less than that at which coal is carried in open hopper cars in Pennsylvania (3 mills per ton-mile), that flour is certainly carried at a loss.

Why should a community which is in the interior and away from water facilities pay frightful freight rates in order that a community on a water route may move its commodities below cost? All "rate differentials" (better called "rate arbitraries"), all comparative rates, all these evils sap our energy.

We have played more or less into the hands of the railroads in the past by creating a ponderous Interstate Commerce Commission, which sits with such great dignity in Washington. To the speaker its top-heavy deliberations often appear as mirror reflections of an imaginary committee which meets for the purpose of discussing carefully how to eliminate or alter a wave in mid-ocean. A commissioner is sent out and the wave is photographed, a bluff is made at measuring it, it is reproduced, a wax model is made of it, and after all data have been collected, the matter is laid before the parent committee, which goes over all the records, finds the center of gravity, buoyancy, specific gravity, has some cuts made, and borings, and determines whether or not it is homogeneous, in order to decide the best and most economical

Mr. Bernhard.

way to smooth out that particular part of the wave, only to find that, after its carefully prepared decision, the wave has disappeared.

It would be very difficult to decide which is the greater evil, the rate systems, the Interstate Commerce Commission, or the conception of transportation by some old-time railroad "pioneers." Unconsciously, the Interstate Commerce Commission has been saturated by that all-penetrating conception of rate-making that the railroads have forced on the public mind and which has permeated America like a chilly fog, blanketing and retarding progress. Although perhaps some members of the Interstate Commerce Commission were willing to remedy this evil, when appointed they are placed in the strait-jacket of the law based on the rate evils, and are not able, laden down with decisions and handicaps, to undertake the David's task of killing this Goliath, even if this Goliath has clay feet.

The speaker has tried to follow the Interstate Commerce Commission, and honestly attempted first to read all its decisions. Then he had to content himself with owning the publications, and now even this has to be given up, unless larger offices are rented to find room for the gigantic library that is turned out of this grist-mill. It is far beyond a shipper, it is far beyond a transportation engineer, to understand that labyrinth. It has become necessary, for a great number of years, to employ special legal experts who devote their lifetime to rates. It is for this reason that every local community, Chamber of Commerce, Board of Trade, Business Men's League, etc., employs traffic managers, traffic commissioners, rate clerks, or whatever their titles may be, for the sole purpose of interpreting rates or trying to get special rates, even if it is at the cost of others—yes, entirely regardless of whether it is at the cost of the United States in general, or any community in particular ("pork barrel" patriotism). It is most customary for one community to try to get the rates of another community raised because they are entitled to a "differential", meaning that, for some reason, born of error and evil, they should be entitled to carry their products into a certain field cheaper than another community. The question of cost of carrying does not enter into consideration.

The proper rate system should be that of terminal charges plus cost of transportation and profit. All railroads should deliver their freight to warehouse doors on sidings, if such exist; otherwise by drays. It is a comparatively simple matter to determine the cost of carrying such freight to and from the proper starting points (freight-yards). Such collection or distribution of freight points to the vital necessity of public ownership of terminals, belt railways, switch-yards, etc. To determine the cost of carrying between terminals is a problem the solution of which is not at all impossible or impractical. In fact, on certainly 90% of the articles carried in America we can determine the cost of transportation very accurately.

Mr. Bernhard.

Of course, the cost of moving freight in certain communities and localities depends on the density of population and on topographical conditions; in other words, if this principle is carried through to its logical conclusion, it will be found that a certain railroad will have to charge a higher rate than a competing line, because the cost over its system would be higher than the cost over others; but, nevertheless, rates should not be maintained below a just level because a railroad cannot get the freight under its correct rates. It is a logical conclusion, and must be accepted on its face, even regardless of results. There is no reason why a shipper in one State should pay a higher rate than cost-plus-profit in order to pay in part for the freight of another shipper elsewhere who pays less than cost-plus-profit. As each community develops, its rates should be reduced.

It is perhaps appropriate to point out: First, that the general statement that freight is carried cheaper in America than in any other country except India is incorrect. It is one of those examples in which statistics lie. The statement is often made that where our cost of carrying is about $7\frac{3}{4}$ mills per ton-mile in average for the entire country, it is, say, in England (before the war), 22 mills; but the fact is that this $7\frac{3}{4}$ mills per ton-mile in average is arrived at by including all the freight, 60% of which is agricultural, forest, and mineral products, a very large percentage of such products being moved in France, England (coastwise), Holland, Germany, etc., by water, and, because these low-paying commodities have been left out of their average, such average cost of freight carrying seems to be higher. If, however, we eliminate these three products from the total freight carried, and then ascertain the average for the remaining products, we will have entirely different figures to compare.

Second, it is equally incorrect to state that in the only place in America where the railroads are governmentally owned, the rate, in cents, is more than the average for the privately owned railroads, in mills. The road referred to is, of course, that in Alaska. The question is, what would the rates have been, under private ownership, of a road working under the existing conditions in such a sparsely settled community? The only fair comparison would be with the cost in New Mexico, and even that would be very much in favor of the New Mexico lines, because they at least can operate the year round without snow interference.

Third, the idea advanced by Mr. Smith, on public ownership but private operation of tracks and terminals, has been followed in Holland for many decades, and has worked very satisfactorily.

Mr. Ray.

G. J. Ray,* M. Am. Soc. C. E.—Mr. Smith seems to know something about the Delaware, Lackawanna and Western Railroad and the Cen-

* New York City.

Mr. Ray. tral Railroad of New Jersey. If he lived along the Lackawanna and had looked into the charter of the old Morris and Essex Railroad, he would have found that he has the right to get a buggy, with proper gauge, and, if he complies with the rules and regulations of the road, run his buggy over the Lackawanna as he desires, for the charter of that road, one of the first railroads built in New Jersey, and one of the first in the country, provided that any citizen of the State should have the right to run his buggy over the railroad between Newark and Morristown. He could go on the tracks at any crossing, so long as his buggy was of the proper gauge, if he lived up to the operating rules and regulations of the railroad.

The trouble is that most of our citizens do not want to look up the facts, and do not want to live up to the obligations after they find out what the facts are.

Most of the railroads, especially around New York City, are endeavoring to give the public good service, and have been succeeding fairly well. The speaker believes that if Mr. Bernhard would further analyze the rates in the case he cites, he would find that the salt that went from Syracuse to Baltimore, from Baltimore to New Orleans, and from New Orleans back to St. Louis, was carried at a less rate than it would cost to ship the same salt by rail a distance of from 100 to 200 miles in Germany. Certainly, if all the facts were given consideration, he would not compare the railroad rates in the United States with those of Germany, and, as far as that is concerned, we are doing business here in America now and not in Germany.

There was one point of Mr. Stuart's paper, brought out by Mr. Kittredge, that might be considered a little further, and that is the control of unproductive expenditures required by the public. During the last 10 or 15 years, the railroad with which the speaker has been connected has been spending a lot of money for the purpose of eliminating grade crossings, as well as otherwise improving its line.

Little does the public realize how expensive the commuter business is for a railroad that is so unfortunate as to have to take care of it. That may seem like a more or less ridiculous statement, but an analysis of the facts will show that the railroad that has to take care of a large commuting business, to and from a large center like New York City, has a very serious problem to handle. Where such an immense amount of business is concentrated, especially during such times as we have experienced in the past two or three years, where the capacity of the roads and the shipping facilities have been taxed to the limit, the commuting business is a very grave problem.

The plant required for such a business is necessarily large, compared with the return, and if one considers the total money spent for equipment, terminals, suburban stations, and other expensive improvements,

which are necessary in connection with suburban stations, such as grade crossing elimination work, street diversions, drainage, and sewers, which are always more or less serious problems, it will be found to be very expensive. Mr. Ray.

It takes an immense quantity of equipment, motive power, and cars. The motive power and cars lie idle for the greater portion of the day, for they are used for only two or three hours in the morning and for about the same length of time in the evening. Furthermore, the staff of employees must be sufficient to operate the maximum number of trains. These employees must be paid a minimum wage, and, from an operating standpoint, it is often a difficult matter to use the men to advantage throughout the greater part of the day.

In many cases the suburban business greatly interferes with handling freight to and from the terminals. It cannot be delayed on account of the freight business, and this adds to the cost and increases the delay in handling the freight.

All things considered, as a rule, it is not a very lucrative business, and is one about which the officers and management are criticised almost every day. Honest criticism is all right, and sometimes is productive of good, but, as a rule, the commuter has but little knowledge of the many problems involved, nor does he fully realize the very heavy expenses connected with it.

In rebuilding the stations and eliminating grade crossings in suburban districts, it is usually difficult to satisfy the local commuter and finally convince the people that they are getting the best that has ever been built. Therefore, such improvements are very expensive and bring very little return. As Mr. Kittredge stated, scarcely one additional commuter can be traced to such improvements.

The speaker believes this to be one of the cases in which there ought to be some definite method of controlling expenditures and providing the necessary funds to pay for the improvements, whether through rates or a proper allocation of the expenses.

Herbert C. Keith,* M. Am. Soc. C. E.—At present much is heard about conservation, a subject deserving the best thought of all men; but much that poses under that name in the Government operation of railroads reminds one of Aesop's fable of the boys stoning the frogs. It may be fun for the Railroad Administration, but it comes too near being fatal to the traveling public. Mr. Keith.

When Mr. Brandeis, now Justice of the Supreme Court of the United States, stated that the railroads of this country could save a million dollars a day by the adoption of such methods as holding trains until sufficient cars—passenger or freight—were filled to make a train using the full capacity of the locomotive, and running the same at such

* New York City

Mr. Keith.

speed as to procure the maximum hauling capacity of the coal, water, etc., consumed, probably few supposed him to be in earnest. Probably no one believed that the public would put up with the inconvenience such methods would entail; but, under the stress of war, the Government has adopted approximately those very methods, making some saving to the railroads (or their administration), but putting the public to an increased expense of many times the saving to the roads. With the duress of war removed, it is not probable that the public would submit long to such an abuse, whether under Government or private ownership or control.

The number of city railroad ticket offices in Greater New York was reduced to five, requiring a large amount of intra-city travel by the public in order to purchase tickets or to engage accommodations. Even in those five offices the number of employees was reduced to such an extent that the traveler had to stand in line a long time before receiving attention. When purchased, the tickets were good only for the day of purchase; and the sale of mileage books became simply the taking of the purchaser's money by the Railroad Administration in advance, with no benefit to the purchaser, either in cash discount or the saving of later inconvenience; for an exchange of the mileage coupons for the regular passage ticket was required when one was to travel, and at the same price.

The new brand of railroad conservation has been shown, also, in scanty maintenance work, both in way and equipment, so that large expenditures must be made soon, or there will be increased expense, delay, and trouble. Only slightly more than half the normal quantity of rails, ties, and other track material was used in renewals in 1918; stations and other facilities were allowed to get badly out of repair, and almost nothing was done in the way of bridge renewals or repairs. A large number of locomotives and cars were purchased; however, they were not credited to maintenance, but charged to the railroad companies as improvements, though, to a large extent, they were misfits for the different railroads, because the microbe of standardization has attacked the Administration and has led to the purchase of the same kind of rolling stock for mountain roads as for those that have spent millions of dollars to reduce their grades to less than one-half of 1 per cent.

In spite of all this scantiness of expenditure for maintenance of way and equipment, operating expenses have increased 40% above those of the previous year, and the traffic has been substantially the same. This increase of expenditure was due largely to increases in wages of those employees organized in the labor unions, and to the high cost of material and general supplies. Executives, engineers, and other technical men have had no share in this increase, but rather have been dropped, or have had their salaries reduced or thrown on the railroad companies (in distinction from the U. S. Administration),

under the plea that their work concerned the conservation of property rather than the operation of the roads. It is amazing that the cost of such work as the preservation of the title to real estate, reports to and meetings of the stockholders, legal expenses connected with the property, and its valuation as required for the Interstate Commerce Commission, was included in the operating expense in the years on which the compensation to the security holders was based, but such charges were not allowed as operating expenses when making payment—or the promise of such—for the actual use of the property. Mr. Keith.

The agreements made by the Government with the railroad companies call for the return of the property, at the expiration of the period of Government operation, in the same condition as it was received. If this is done, it will require, before that time, for the maintenance of way and equipment, the expenditure of a larger sum of money than the $500 000 000 "Revolving Fund" already used up in the operation.

For years the railroads have been asking permission to make a small increase in rates, but were refused because the Interstate Commerce Commission denied the necessity for such increase to provide a reasonable return on the investment of the stock- and bond-holders. At the same time the Labor Boards awarded increase after increase of wages. A short time before the Government took the roads, a small increase in rates was granted, but this was too late to affect materially the earnings for the period used as the basis for the return to be paid by the Government to the owners for the use of their property. Soon after the U. S. Administration took charge, passenger and freight rates were increased, so that receipts for 1918, for approximately the same amount of business, increased 22% above those for the previous year, but, in spite of this, because of the increased expenses, as previously stated, the net earnings were reduced to such an extent that the $500 000 000 "Revolving Fund" is exhausted, and the Administration asks for an additional $750 000 000 to make up the deficit of what is required to be paid to the railroad companies according to the agreements—even as the Director General himself interprets them.

Demand is now made for five years' continuance of Government operation, because the time has been too short to determine the advisability of Government ownership. One naturally asks "Were the railroads taken for the purpose of aiding in prosecuting the war, or was that only camouflage to conceal Government ownership?"

The subject of rates has been mentioned. It is a very interesting fact that, before the war, in normal times, the railroad companies of the United States paid higher wages and salaries than any other country in the world, while, at the same time, the railroads of this country gave cheaper transportation and better service for both passengers and freight than any in the world, with the exception of India, where rates

Mr. Keith.

were a trifle lower, but where the pay of laborers was hardly more in cents than it was in dollars in the United States. It is an interesting fact that on the only Government-owned railroad of the United States, prior to the war, the freight rates were more, expressed in cents per ton, than the average of those of the privately owned railroads of the country, expressed in mills.

In connection with Mr. Ray's remarks about the rights of the Delaware, Lackawanna and Western Railroad, it is interesting to note that the charter of the Boston and Lowell Railroad, granted June 5th, 1830, authorized the company to erect toll-gates at any and all points where it might desire, and to collect the tolls thereat, evidently with the idea that anybody could harness his horse or mule to his car and use it on the railroad. Just before the bill received its third and final reading, an amendment was made, providing that, in case the experiment then being conducted in England should prove successful, the railroad might use steam.

Mr. Williams.

CLEMENT C. WILLIAMS,* M. AM. SOC. C. E. (by letter).—This paper is timely, and contains many interesting and valuable suggestions. Dissertations of this sort will assist greatly in securing an economically sane and sound solution of this problem, which affects our industrial organization so vitally.

As the author states, investments in railway securities should be protected, but they should also be removed from the realm of speculation and manipulation. Investments in railroads should be nearly as stable as those in National banks and in Government bonds, if the conditions controlling them were properly adjusted. Such an adjustment, in order to be adequate, must be based on a correct conception of the relative interests of the railway corporations, their employees, and the public, and of the physical conditions affecting the operation of the railroad lines. For example, the author mentions the necessity of controlling unproductive expenditures, such as grade-crossing elimination. Based equitably on an economic distribution of benefits, all such expenditures as grade-crossing elimination, beautification of station buildings and grounds, etc., should be borne largely by the public instead of by the railroads, for the benefits accrue to the public rather than to the railroads or their patrons, and the burden should be placed where the benefits are received. As the benefits of beautified stations and grounds accrue to the local public, their cost should be assessed against them rather than against the railway corporation or against the traveling public through increased rates.

Moreover, such a distribution of expense, in addition to decreasing operating costs, would tend to correct abuses which are of frequent occurrence, for a municipality would think twice with regard to the

* Lawrence, Kans.

necessity of grade-crossing elimination or track elevation before passing an ordinance requiring such work, if it had to pay the bill. Mr. Williams.

This theme of joint responsibilities and benefits leads to a consideration of the entire railway question on such a basis, with a view to ascertaining the proper reconstruction programme as it may pertain to the railroads.

In the reorganization of the railroads, obviously, the aim should be to eliminate the evils of the previous régime of controlled private ownership under quasi-competitive conditions, retain its good features, and add such further provisions as will render the railways better adapted to the needs of the nation. The outstanding general defects that existed in our railway administration previous to the leasing of the lines by the Government are:

1.—Stock watering and other phases of unsound finance;
2.—Lack of stability in railway securities;
3.—Inefficiency and waste through lack of co-operation due to competitive jealousies;
4.—Cumbersome, conflicting, and ineffective control by State and Federal authority;
5.—Disregard of the rights of the public in matters pertaining to service and in labor controversies.

The meritorious features of private operation have been demonstrated to be:

1.—Economy and efficiency in operation resulting from placing actual responsibility in the operating officers, rather than having them act merely as agents for an extraneous bureau;
2.—The initiative that results from local responsibility;
3.—The gain derived from direct dealing and the absence of red tape that seems to be an inevitable concomitant of governmental administration;
4.—The freedom from harmful interference in administration due to the influences commonly designated as "politics."

In addition to avoiding the evils and retaining the advantages of private operation and ownership listed above, the purpose of any new measures should be to:

1.—Effect a more complete realization of the unity of interests among railway companies, their employees, and the public;
2.—Give adequate Federal control over rates and financial policies;
3.—Adjust the railway systems and groupings on the basis of optimum service to the nation as an entity in meeting its transportation requirements, making provision for pooling and direct routing of traffic.

Mr. Williams.

So far, the only solutions of the railroad problem commonly advocated are a complete return to private ownership and operation after the expiration of the Government lease, on the one hand, or ownership and operation by the Federal Government, on the other. Inasmuch as either of these arrangements is admittedly open to serious objection, it may be wise to cast about for another scheme, with a view to obviating the defects and securing the desirable features of both. There is no law or reason compelling one of these plans to be adopted *in toto* exclusive of the other. Certain situations have arisen in the railroad world within the past few years that may suggest a general scheme for the solution of the difficulty, although the final adjustments would doubtless have to be made as time might reveal the need.

The experiences under the past mode of administration and under present Governmental operation, as well as some other experiments that have been tried in the field of railway transportation, seem to indicate that the plan which is most likely to secure the objectives mentioned above is a median course between private ownership, on the one hand, and Government ownership on the other, a plan that would bring together into a partnership the parties involved, and harmonize and unify their interests. The principles underlying such a partnership are suggestive; hence, an outline of the main features may be considered with profit.

In the first place, the railways would serve the needs of the country most effectively if they were organized permanently in a comprehensive manner, designed to meet the transportation requirements of the whole nation as an entity. A state of development has now been reached in which the centers of population are fixed, the character and location of the main fields of industry are determined, and, consequently, the transportation requirements of the future may be reckoned chiefly as a normal growth and development along the lines already established. It is time, therefore, that a comprehensive transportation system is designed to meet the conditions as they exist now and may be predicted with reasonable certainty for the future. An effort is all the more opportune at this time, when the railway situation is to be made over, when, in a sense, "the cloth is uncut", for now the railways can be regrouped boldly according to the economic requirements of the country, unhampered by any existing artificial financial alliances. The leasing and operating of the railroads by the Government has given a new point of departure and an opportunity for a comparatively fresh start in formulating a railroad policy, and advantage should be taken of this fact to plan boldly and without prejudice for fitting the railway transportation to the economic and industrial situation.

In order to secure the advantages of direct routing and of pooling equipment and similar methods of operation which the present régime

Mr. Williams.

has demonstrated to be advantageous, the railways should be grouped permanently into regional systems and incorporated under holding companies. Such regional systems should follow the natural traffic groupings, instead of artificial and arbitrary regional assignments as made by the Director General, or even those of the Interstate Commerce Commission. Such a plan, of course, would be based on a thorough statistical study of the natural traffic routes, and would necessitate a carefully studied classification of roads into trunk lines as main arteries, subsidiary lines, and feeders, with a view to construction, maintenance, and operation as such.

The major financial policies of such large companies as just suggested could best be regulated by the Federal Government holding a controlling share of the stock. Under such an arrangement, no stock watering or other fraudulent procedure could be carried out without Government sanction, and hence action by the Government could be secured before the harm was done. One of the defects inherent in the previous methods of governmental control has been that the nefarious schemes of stock manipulation were completed and the damage to securities was done before the Government was aware of what was happening, and before the governmental machinery could be got into motion to prevent it. The result has been that it was a case of "locking the barn door after the horse was stolen." Laws were enacted which illegalized a repetition, but did not prevent a similar thing from being done in a different way. With the Federal Government holding a controlling interest in the stock, however, it would necessarily be apprised of any contemplated action of this sort.

A ready means of stabilizing railway securities under such a scheme of organization would be to issue the Government's stock as junior to the privately owned stock, making it the first to meet assessments and the last to receive dividends. Perhaps the Government might hold practically all the common stock. Such a preferential payment plan would establish railway securities on a firm basis without the necessity of guaranteeing returns on private investment by the Government. Making the stock owned by the Government junior to the privately owned stock would work no injustice to the Government (provided, of course, that the limits of preference were equitably adjusted), for the Government, as a social community, would receive benefits in other ways than in returns on stock, *viz.*, reliable transportation, diminished fluctuation in industry, wider range of industrial and social activity, increased land values, etc. The information obtained by the Division of Valuation of the Interstate Commerce Commission would be of service in such a reorganization. A board of directors, under such an arrangement, might properly be composed of representatives of the Government, representatives of labor, and representa-

Mr. Williams.

tives of private capital. Beyond the board of directors, the operation of the lines should be entirely under private control.

Much has been said of the rights of the public, as affected by the railways, in regard to the character of service rendered, inconvenience and loss during labor controversies, etc., but there has been no apparent inclination on the part of the public to share the responsibilities of operation. This attitude is not in accord with sound economics, for rights and responsibilities should go hand in hand. By entering into a partnership, the public could control the general policies of the railways and thus protect their rights, and, at the same time, share their proportionate responsibilities. For a similar reason, employees should be encouraged by some means to hold railroad securities.

Examples of organization similar to this plan may be mentioned as partial illustrations of the principle involved. The National Railways of Mexico, constituting somewhat more than half of the total mileage of the country, are organized on a basis similar to that just described. Of the total issue of stock of $224 000 000, the Mexican Government owns $115 000 000, which includes all the common stock, about one-third of the first preferred, and about one-sixth of the second preferred. The National Railways cover practically the entire country, and form a network articulating with the other railways. All shares of the National Railways have equal voting rights, although the preferred stock has the right to elect five directors separately. The Mexican Government guarantees the principle and interest of the general mortgage bonds, although any payments made by the Government under this guaranty are to be repaid out of net profits otherwise distributable as dividends with interest at 4%; and this repayment takes precedence over all dividends except on first preferred stock. The Government has come to the aid of the railways about six times under this arrangement. All dividends are tax exempt. The result is that the railways are about the most stable feature of that country.

A similar scheme of organization exists in New York City whereby the city and the electric railways within its borders have formed a partnership for further providing the city with transportation facilities. In financing the Dual Subway plan, the City of New York is to furnish $171 000 000 and the railways involved $166 000 000 for the extension of the subways. The stock owned by the private parties is to receive preferential payment of dividends, taking precedence over municipal stock. This scheme is economically sound, for the city as a community receives benefits from possessing an adequate transportation system which may be properly set off against the dividends received by private stockholders.

In such a scheme of organization as described above, the railways become essentially "operating companies" rather than "owning companies." Under the present governmental administration, a definite

distinction has been drawn between the officials representing the corporate interests and those concerned with operation. Moreover, Government-owned munition plants, shipyards, and other properties have been placed in the hands of "operating companies" during the past two years. The idea, therefore, of an operating company is not entirely new, but has a more or less recognized status in the present industrial organization. Private operation has been demonstrated by experience all over the world to be superior to Government operation, provided the abuses of private ownership can be eliminated, and should be retained, therefore, if possible. Mr. Williams.

Summarizing: The present situation is such as to afford an opportunity for re-grouping and reorganizing the railways of the country according to a comprehensive design adapted to the economic needs of the country as a whole, and steps should be taken to obtain expert determination of these economic requirements as related to the railways. To avoid the defects of private ownership, and, at the same time, obviate the inherent faults of Government operation, the most promising arrangement seems to the writer to be the formation of a partnership between the Government and the railways. This appears to be preferable to the more or less inimical control on the part of the Government, which has tended to stifle development in the past, and to discourage legitimate investment in railway securities. Moreover, it is only by an appreciation of the community of interest of the public, and of the railways, that the economic welfare of the country as affected by the railways can be realized.

C. A. Morse,* M. Am. Soc. C. E. (by letter).—The writer would like to call attention to one of the statements in this paper, which he fears is too sweeping and may lead to misunderstanding. Mr. Stuart states: Mr. Morse.

"Some of the things which were ordered with propriety during the war, which should be undone at once and not five years hence, are: * * * an immediate abandonment as a national effort of an attempt at standardization of all manner of railroad necessities and methods."

This might lead to the impression that there was under way, at present, an attempt at standardization of all manner of railroad necessities and methods, which is not the case.

There were numerous committees at work during the war, under the War Industries Board, in an attempt at general standardization. Their work, however, was shut off as soon as possible after the signing of the armistice, and no such national effort at standardization is being undertaken by the United States Railroad Administration.

There are, however, without doubt, many things that could be standardized with advantage to every one. One of the most important

* Chicago, Ill.

Mr. Morse. of these is standardization of rail sections, and of the location of bolt-holes in rails. At present, there are nearly fifty different sections of rail between 70 and 136 lb. per yd. Notwithstanding the fact that the American Railway Association and the American Railway Engineering Association have recommended as standard only seven sections between these limits, practically every railroad has a different spacing for bolt-holes, and there is a great variation between the location of the center of bolt-holes as regards the neutral axis of the rail. Standardization of rail sections and of the drilling of rails is a very desirable thing to accomplish.

The fact that the standardization of methods and appliances has been found to be a source of economy on the larger railroad systems, thereby reducing the stock of materials necessary to be kept on hand, plainly indicates that, could this be extended to the railroads of the country, it would mean economy in manufacture, which, in the end, would mean reduction in the cost to the railroads and the public.

One of the dangers in standardization, however, is the idea expressed in the last portion of the paragraph in the paper from which the foregoing quotation was made, that standardization to some people implies that an approved standard means a perfected article. There is no reason why standardization should limit, in any degree, efforts to improve devices or to adopt new standards at any time when an improvement in a standardized article has been perfected.

The writer agrees with Mr. Stuart that wholesale standardization is a dangerous proposition, but he feels that standardization used with judgment has a broad field for economy.

Mr. Whinery. S. Whinery,* M. Am. Soc. C. E. (by letter).—It is clear that public opinion is strongly opposed to exclusive ownership and operation of the railroads by the Government; and there is a strong body of opinion opposed to their return to private ownership under the same status as before the war. Some scheme seems to be generally favored under which ownership and operation shall be in the hands of the private or corporate owners, with adequate Governmental power of control in certain important respects. There is, however, much difference of opinion as to the extent and details of the powers which the Government should exercise.

The difficulties likely to be encountered in a dual scheme operated by independent parties would doubtless develop from the diverse and often antagonistic views and interests of the three parties involved—the owners, the Government, and the public.

If some plan could be evolved under which these three interests could be unified and harmonized to a great extent, with a competent

* New York City.

and disinterested tribunal to adjudicate such differences as might arise, a distinct approach to the ideal would be attained. Mr. Whinery.

Can a practical working project of this general character be devised?

The question is worth consideration and discussion, and, as a contribution to discussion along such lines, the following suggestions are submitted:

The controlling features of the scheme suggested are:

1.—Joint ownership of the railroads by the corporate owners and the National Government;

2.—The operation of the roads by the corporate ownership;

3.—Control by Governmental agencies of certain important functions, such as general financing, basic policies, co-ordination of operating plans, and supervision of rates;

4.—The creation of a tribunal having ample authority to adjust differences that might arise and be referred to it.

A brief outline only of the suggested scheme can be here undertaken. If the general project has merit, the details should not be impossible, and perhaps not very difficult to work out.

1.—Congress should enact the necessary legislation, requiring all railroads to be incorporated under the United States Government and embodying all necessary provisions for carrying the plan into effect. Among these would be:

2.—The creation of a tribunal of mediation and control, of last resort (except to the Supreme Court), to which questions not otherwise adjusted could be referred and adjudicated. This tribunal might be a commission like the Interstate Commerce Commission, with enlarged but well-defined powers, or a Department of Transportation, the head of which might be a member of the Cabinet.

3.—The acquirement by the National Government of a safe controlling interest (ownership) in the leading railroad systems and important independent lines of the United States, by the purchase of shares of stock in the open market, or, if necessary or advisable, by the exercise of the right of Eminent Domain. In the latter case, shares should be taken from shareholders in proportion to their holdings over, say, five shares; the purpose being to preserve as wide a distribution of privately owned shares as possible.

4.—Unless conditions should arise making it necessary to do otherwise for the public good, the secondary or less important roads would not be included in the scheme; that is, they would remain wholly in the ownership and control of their corporate shareholders. These less important roads have been, and would continue to be, influenced, directly or indirectly, by the policies and practices of the more important lines with which they connect.

Mr. Whinery.

5.—Provide that the boards of directors or controlling bodies of the jointly owned roads or systems shall each consist of an uneven number of directors, a bare majority of whom, to be known as Government directors, shall be appointed by the President of the United States, while the minority shall be elected in the usual manner by the private shareholders—the Government shares not to be voted at such election.

6.—The functions and duties of the board of directors to be divided into two general classes:

a.—All matters in which the Government may properly exercise supervision and control; such as capitalization, general financing, controlling policies, combinations, consolidations, competitions, apportionment of earnings, and the making and general supervision of rates; also, the appointment of presidents or chief executives.

b.—All matters relating to the practical operation and management of the roads, including construction and maintenance of road, equipment and plant generally; the appointment and control of all subordinates and employees, the primary making and adjustment of rates, schedules of operation, etc., and generally of all matters not specifically named in Class *a*.

7.—In all matters coming under Class *a* each member of the board would be entitled to one vote, and all members of the board would stand on an equal footing. In all matters coming under Class *b* each Government director would be entitled to one vote and each corporate director would be entitled to two votes. The result would be that in all Class *a* matters the Government directors, being in the majority, would be able to exercise control, while in all Class *b* matters the corporate directors, by reason of their dual votes, would have a controlling voice.

8.—The Government directors should be appointed for life, subject to removal for cause, and should devote their whole time to their duties. Each should serve on the directories of a number of different roads in the same region, or having common, related, or competitive interests, thus making the boards, in a measure, "interlocking" in character.

9.—Any differences that might arise between the two classes of directors, which they were unable to adjust, as to their respective rights or duties, or the interpretation of laws, or general policies, would be referred to and decided by the supreme tribunal, subject to review by the Supreme Court.

10.—The principle that railroad rates shall be such that the shareholders shall receive a fair and just return on their investment, should

be recognized and made mandatory by the basic law. Rates should be fixed and adjusted by the directories of the roads subject to review and modification by the tribunal, wholly independent of Congressional action; but earnings or dividends should not be guaranteed by the Government. Mr. Whinery.

11.—State or local commissions or officials should be authorized by the law to exercise, only, usual or necessary police powers and local control in purely intrastate questions. Their status with reference to the railroads would be substantially the same as in the case of navigable rivers.

The enactment of the necessary and appropriate legislation by Congress to carry out this—or in fact any other—general railroad plan, would be a work of prime importance, requiring careful study and foresight. Its provisions should be definite and clear, as full and detailed as practicable, but, at the same time, permitting such latitude and elasticity as would allow and foster enterprise, initiative, and efficiency in the management of the properties.

The Government directors, of course, should be men of high character and special fitness for their duties, and the method of appointing them reasonably might be expected to secure such. Life appointment, as in the case of Justices of the Supreme Court, should remove them from the sphere of political influence.

Some of the merits and advantages of such a project, properly administered, are obvious, but they may be briefly outlined:

1.—The joint ownership, and resulting common interest in the properties, of the corporate shareholders and the Government, representing the interests of the public, would tend to remove or neutralize the spirit of distrust and antagonism that has been, particularly in recent times, one of the most discouraging features of the railroad situation. The participation in the management of able men, representing the interests of the public, with no private, partisan, or political ends to serve, with power to control in all matters in which the Government or the public it represents might properly intervene, should secure in large measure the respect and confidence of the people.

2.—The practical result would be to secure all the benefits of Government ownership combined with those of private operation.

3.—The interlocking feature of the boards, by which the Government directors would serve on and be distributed among the directorates of a group of roads with related interests, would give them opportunity and facilities to acquaint themselves with all the facts and conditions of the situation, and they would be in a position to permit or direct such combinations and adjustments of routes, rates, interchanges of business, and apportionment of earnings, etc., as might seem just, and best for the interests of all concerned.

Mr. Whinery.

4.—Although the necessary investment by the Government would be large, the means could be provided by the issue and sale of bonds at a comparatively low rate of interest, with the certainty that its share in reasonable dividends would greatly exceed the interest charges. A surplus should result which would be a dependable source of large revenue for public purposes, to that extent relieving general taxation of the public.

5.—The purchase of the Government interest in the railroads would release a large amount of capital to the corporate shareholders, to be reinvested in other industrial or business enterprises.

6.—Such a project, when once properly organized and installed, should be permanent in character and results, subject only to such minor changes and improvements as experience might prove to be necessary or desirable.

7.—One of its most important results should be to remove effectually our troublesome railroad problems from the sphere of partisan politics.

It may be objected to this plan that boards of directors composed of two distinct classes of men, representing different interests, would not be likely to work together harmoniously; but, in fact, they would represent the same and not different interests, as the purpose of all would be to secure and promote the efficient and successful management of the railroads for the benefit of all the shareholders as well as the public. If, for instance, rates and earnings were made too high, the public would share in the surplus through such lower general taxation as the Government earnings would permit; if too low, the public would be called on in the same way to share a part of any resulting losses.

It is obvious that, if such a scheme proved to be practical and successful in the case of the railroads, it might, with some modifications, be applied to the control of the great industrial businesses that are commonly regarded as monopolistic, and in need of some governmental regulation.

Mr. Stuart.

FRANCIS LEE STUART,* M. AM. SOC. C. E. (by letter).—So much has happened since the paper was published, so much wandering of ideas, so many plans discussed, so many bills proposed, and so little accomplished that in closing the discussion it is impossible to more than touch on some of the fundamental aspects of the case.

Private capital, with expectation and hope of gain, built the railroads which have made the United States great. The amount of capital so invested can be dissipated by public policy, or the mistaken judgment of either capital or labor, but if so dissipated, new capital for further progress in this line will have to come from taxation, and will be spent on a partisan political basis instead of on the financially sound basis of an expected return on the investment.

* New York City.

Mr. Stuart.

Government ownership, in spite of propaganda in its behalf, seems to be still more impractical and unthinkable than it did in January, 1919. The readjustment demands of the times urgently require that the Government return the roads and set in motion this return and the relinquishment of other abnormal functions that it assumed during the war.

The railroads have lost their credit, and before they can stand alone their financial standing must be rehabilitated by a law enacting broad constructive policies and by the extension of Government credit, in order that money may be procured until such policies actually show that they are beneficial. The only way that private investors can be induced to put their money into the jeopardy that their past investments in railroad securities have been placed, will be the assurance and visible evidence of a continuing policy which will give a fair return on the average railroad, efficiently operated.

Innumerable plans have been presented and discussed, and a number of bills have been proposed, for solving the railroad question. They all shed light and may be illuminative, but, individually, the writer thinks that if Congress will turn the roads back with a proper emergency rate and with the necessary protection of Government credit, the next few years could be well spent by Congress, not in passing new bills, but in repealing a large part of the existing law on the railroad subject and in codifying the remainder so that it can be understood by the public.

The fundamental principle of American business, in fact of all business, is to charge what the traffic will bear, be it a railroad or a corner grocery shop, and supply and demand governs the application of this law. The Government has found it necessary for the benefit and protection of the public to create general laws to regulate business, which are applicable also to the railroad business. On account of the public character of the railroads, in addition to such general laws of the land, there should be special laws, governing safety, preventing discrimination and unreasonableness of rate, and requiring reasonable service. The greatest factor in making such laws really effective is the preservation of competition and reward for initiative. With these incentives many new laws now in existence could be repealed.

If the prohibitive laws which now prevent the consolidation of railroads under certain conditions were modified, a natural consolidation of a number of properties would follow, a consolidation based on competitive reasons and on business standards of a return on the investment so made. The writer doubts whether a Federal consolidation of all railroads into one, or two, or ten, or any stated number of systems, not based on competitive reasons, would eliminate more than $200,000,000 of duplicated capital investment and, for the sake of eliminating this comparatively small amount, we would be bartering

Mr. Stuart.

away our birthright of competitive service on $20 000 000 000 of capital invested in our railroads.

On account of the difference in conception and in the conditions of the various railroads, some will prosper on a given rate while others will fail, but, in the writer's opinion, it is unconstitutional—or should be—for the Government to guarantee a fixed amount of return on the investment in railroad securities and not do the same thing for the coal business or the wheat business or for any other industry dealing with the necessities of life—any of which could be made bankrupt of credit by the same treatment which has been given to the railroads in the near past.

It also seems to be unconstitutional—or it should be—for the Government to give to labor or to capital or to any class or community any amount earned by the properties over and above a fixed return on the capital investment. If any such profit should legally accrue to the Government, it should be spent by the legislative powers for the benefit of all the people.

The reward for initiative and sound judgment should be maintained and the average railroad allowed to prosper as a right and not by sufferance. Railroads must pay their way from day to day, and become a reasonably prosperous self-sustaining factor in our communities, or they will become a dangerous political canker.

The political temper of these readjustment times, combined with other causes, seems to have prevented constructive work by either political party, and the results at least make it doubtful if it is wise for Congress with its present mental attitude to try to pass remedial legislation under the present abnormal conditions of capital, labor, and Government functions.

Why not, then, pass simple legislation, providing for an extension of credit and a fair emergency rate? The roads should be returned to their owners at once and as an emergency act the railroads should be extended credit by the Government and given a fair emergency rate to help them over the readjustment period through which the country has to pass.

A fair emergency rate, in the writer's mind, is the present rate properly increased so that when applied to the units of transportation making up the revenue tonnage of all the railroads, it would in the aggregate pay operating expenses and leave an amount over and above the gross operating expenses of all the railroads to be applied as remuneration for the capital invested in all the railroads, such remuneration to be in keeping with the current value of money and be divided among the roads as earned by service.

The Railroad Administration has been in charge of railroad accounting figures, and a percentage to add to the present rate could easily be found. There should be no discrimination in the application of

such an emergency rate. Each road would earn as much of the gross amount as it could in competition with the other roads and as a reward for efficient service, and yet American competitive business methods and standards of pre-war times would be maintained and the result would be fair to the greatest number of the population. Immediate action is needed, and it does not require months to establish such a rate, for if the rate should prove too high in a few years, and if the political feeling of the time should require it, a graduated excess profit tax as a post-war measure could be applied to the more prosperous roads, until during normal times a more equitable rate could be found. Such excess profit tax could be used, as all similar taxes should be used, for a direct reduction of the war debt. Mr. Stuart.

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Paper No. 1447

GROUTING OPERATIONS, CATSKILL WATER SUPPLY*

By James F. Sanborn† and M. E. Zipser, Associate Members, Am. Soc. C. E.

With Discussion by Messrs. Robert Ridgway, J. Waldo Smith, Lazarus White, John P. Hogan, Francis Donaldson, Waldo C. Briggs, A. J. Sackett, Andrew A. Cohill, A. J. Mayell, Milton H. Freeman, C. L. Riker, Jr., Benjamin A. Howes, Theodore Belzner, and M. E. Zipser.

Synopsis

Grout was used extensively in the structures of the Catskill Water Supply System for filling spaces behind the lining of tunnels, for cutting off the flow of water into shafts and tunnels by grouting the water-bearing fissures, for solidifying the foundations and stopping the flow beneath dams, and for various other purposes. The object of this paper is to give a summary of the experience and results obtained in the use of grout in the construction of the aqueduct, dams, and other structures of the Catskill Works, with the hope that it may prove valuable to those undertaking work involving grouting.

Grouting was one of the important operations in the construction of the pressure tunnels of the Catskill Aqueduct. These tunnels, forming about one-third of the total length of the Aqueduct, are subjected to great hydrostatic pressures when in service, and it was essential

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to make them tight against outward leakage. To prevent the escape of water, grouting was depended on to back up the concrete lining and to close all open seams in the rock adjacent to it. The Aqueduct has been in service for two years, and the slight and practically negligible loss of water from the pressure tunnels is evidence of the complete success of the methods adopted in their construction.

To prevent the percolation of water through porous zones under the Ashokan and Kensico Dams, extensive grouting was done, the grout being forced into the rock seams under air pressure through holes drilled into the ledge rock underneath the dams. To cut off the leakage which had developed underneath the foundations of the two earth dikes at the Ashokan Reservoir, and to improve the foundations, holes were drilled through the core-walls and rolled embankment into the rock and grouted.

Grout was also used to cut off the flow of water during shaft sinking and tunnel driving, in order to permit the work to proceed more rapidly and reduce the expense of pumping. Considerable grouting was also done in connection with repairs in parts of the Rondout, Breakneck, and City Pressure Tunnels, where the lining had been ruptured when the tunnels were subjected to working pressures. The breaks in the lining were due to the yielding of the rock when the test pressures were applied, and grout was injected to fill the cracks in the rock surrounding the tunnel.

Other uses for grout on the Catskill Aqueduct included the lining of large steel pipe lines, the grouting of stretches of poor ground in grade tunnels, the filling of the expansion joint openings in the cut-and-cover aqueduct, and for other purposes. In some of these cases, the grout was not forced in by air pressure, but was placed by pouring or by means of a hand pump.

Grouting in Pressure Tunnels.

The utilization of the pressure-tunnel type is one of the notable features of the Catskill Aqueduct. There are nine pressure tunnels, aggregating 35 miles in length. These tunnels were driven in rock, deep beneath the surface, and were lined with concrete averaging from 18 to 24 in., and more, in thickness. They have a finished inside diameter varying from 16 ft. 7 in. to a minimum of 11 ft. The depths of the tunnels below the surface vary from 100 ft. to a maximum of more than

1 100 ft., the latter being the depth at the Hudson River crossing. The hydraulic gradient at this crossing is more than 1 500 ft. above the invert of the tunnel. The pressure-tunnel type was adopted only after extended study, as there was no precedent for such tunnels subjected to such high unbalanced heads.

It was the opinion of some authorities that the concrete lining could not be made tight enough against ground-water heads. It was also feared that where the rock was porous and water-bearing, excessive leakage might develop when the tunnel was put in service, and that the application of pressure in the case of jointed and fissured rocks might even cause movement of the rock and rupture of the concrete lining, with resultant outward leakage. Great reliance, therefore, was placed on the use of grout to make the tunnels secure and tight against all leakage.

Design of Pressure Tunnels.—On account of the heavy internal hydrostatic pressures, it was deemed advisable to place the tunnels under a heavy cover of sound rock. This cover is not less than 150 ft., except in the case of comparatively light heads. On account of the external ground-water pressure on the tunnels when they were under construction, or at any time when they are out of service, the circular shape was adopted, and the lining was made thick enough to withstand both the ground-water head and the grouting pressure necessary to overcome frictional resistance and ground-water head. The concrete for the lining was proportioned, mixed, and placed so as to be practically water-tight. The concrete lining, in addition to preventing outward leakage, also provides a smooth coating to produce a high coefficient of flow. It also prevents the falling of rock and makes easier the cleaning of the tunnel.

As it was practically impossible in lining the tunnel to pack the concrete tightly against the irregular surface of the rock in the roof, grout was forced in to fill all the voids over the arch and back of the lining and to transmit the internal pressures to the rock. It was expected also that the grout would be effective in closing the seams and crevices in the rock surrounding the tunnel by impregnating any porous, water-bearing strata so as to cut off all subsequent outward leakage.

Conditions Encountered During the Excavation of the Pressure Tunnels.—Before the contracts for the pressure tunnels were let,

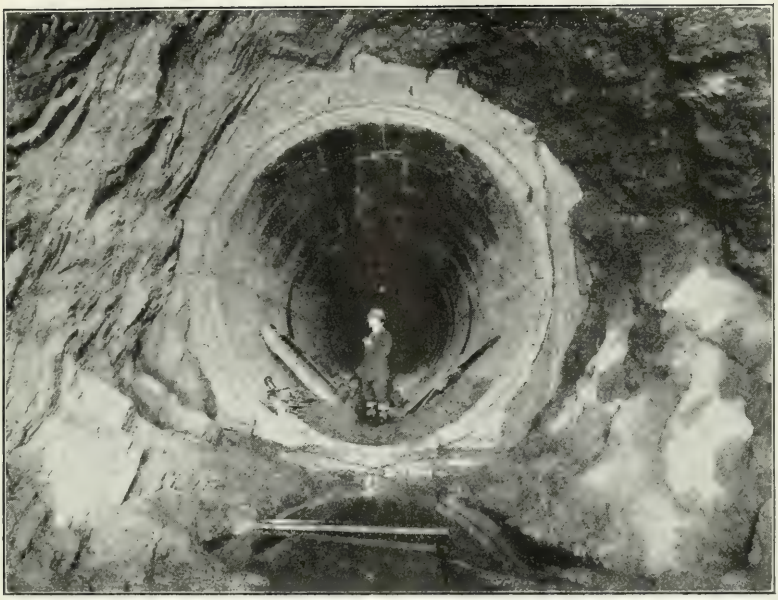


Fig. 1.—Completed Pressure Tunnel in New York City.

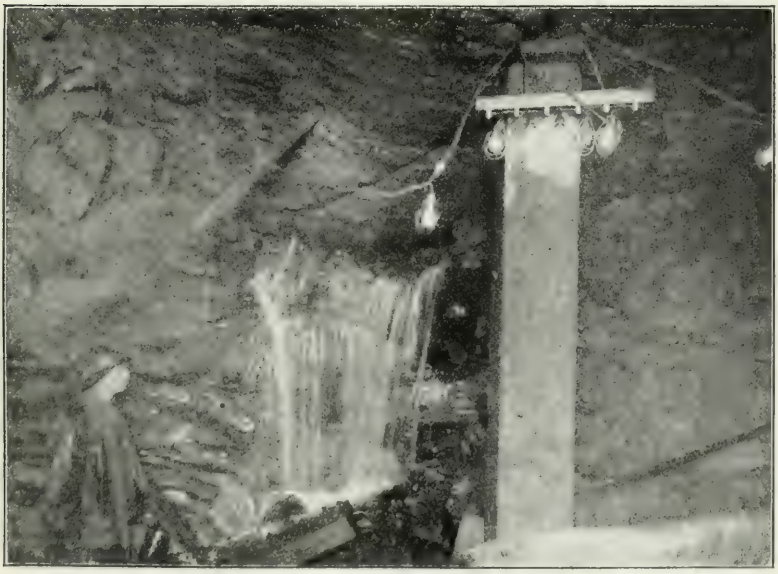


Fig. 2.—Wet Heading in City Tunnel, South of Shaft 4.
A Flow of 200 Gal. per Min. Cut Off by Grouting.

extensive borings were made to develop the rock profile in the valleys to be crossed and to ascertain the kind and condition of rock to be encountered during the driving of the tunnels. The tunnels penetrate a wide range of geologic formations, the rocks varying in character from a soft shale to hard quartz conglomerate. The most important quality, however, from the standpoint of water-tightness of the tunnel, was the soundness and tightness of the rock to prevent outward leakage.

In stretches where the tunnels passed through faulted and crushed zones, and the rock was either unsound or structurally weak, steel ribs and lagging were used to support the roof. At these places the space between the steel lagging and the rock was packed with stone, the dry packing being subsequently grouted through 2-in. pipes set in the concrete lining and extending into the dry packed space.

Although the pressure tunnels were excavated several hundred feet below ground-water level and under rivers and streams, there were only a few stretches where an excessive quantity of water entered during the excavation. In many stretches the tunnels passed through blocky ground intersected by joint planes and crevices; the openings in the rock at these places were often found to be filled with deposits of mineral matter, and little or no water entered the tunnels through such openings. Several of the tunnels passed underneath glacial gorges, and the drift overlying the rock formed a dense impervious blanket against the inflow of water. With the exception of the wet stretches of tunnel, to be described in detail later, no cases of inward leakage occurred during the excavation exceeding 150 gal. per min. per mile of tunnel. In the 18 miles of the City Tunnel, the maximum quantity of water which entered the tunnel, was 137 gal. per min. per mile.

The geological conditions in the tunnels were given careful consideration, as the structure of the rock, the nature of the openings, and other conditions influenced the method of grouting. Underground water circulates in the rock through the intersecting joint planes and seams formed by bending, folding, and faulting due to disturbance of the earth's crust. Stratified rocks are often found with alternating hard and soft layers, some of which are quite porous, permitting free circulation either through the body of the rock or along the bedding planes. Vertical planes of fracture are of frequent occurrence in stratified rocks. In the Helderberg limestones large solution cavities

or mud seams were encountered, requiring special precautions in making the tunnel secure and providing against subsequent outward leakage.

All inflowing water from the rock seams was taken care of before placing the concrete lining by collecting the water and leading it through pipes set in the concrete, as will be described later in detail. Subsequently, the water-bearing fissures were grouted through these pipes. In many cases, the inward leakage during excavation gradually diminished and often stopped after some interval, due to the discharge of accumulated water in the seam, but the points of inward leakage and the areas of general seepage were carefully noted when the excavation was first made. The locations of wet seams were recorded, and provision was subsequently made for grouting at these places, as it was realized that when the tunnel was in service, water could escape through the same channels whereby it had entered.

Grouting the Rock Seams During Tunnel Driving to Cut Off Leakage Into the Tunnel.

Where large flows of water are encountered in driving a tunnel, a loss of progress invariably results, and very often the work is shut down until additional pumps are installed for handling the water. Obviously, it is of great advantage, where the inflow is very heavy, to reduce or cut off the leakage, if possible, before the heading is advanced. As a rule, however, superintendents dislike to stop excavation and grout the wet ground, preferring to continue the driving, if possible. Several efforts were made on this work to cut off the water by grouting the rock during excavation.

Wet Stretch, Rondout Tunnel.—In driving up a 15% incline north of Shaft 4, in the Rondout Tunnel, a faulted and crushed zone in High Falls shale and Binnewater sandstone was encountered, yielding a considerable flow of water charged with H_2S gas. To shut off the water, an attempt was made to solidify the rock in advance of the excavation by grouting through the drill holes in the heading which flowed water. These holes yielded a flow of about 300 gal. per min. They were closed with pipes and grouted, but the high pressure used—200 lb. per sq. in.—blew off 3 ft. of the soft shale in the face of the heading. Additional holes were drilled around the periphery of the tunnel and grouted. Although the leakage was somewhat reduced, the

flow increased with the advance of the heading, as additional water-bearing seams were encountered.

On account of the heavily faulted and crushed condition of the shale, which was intersected by wide and narrow seams, it became apparent, after further efforts, that grouting could not be depended on to reduce the leakage materially, and it was then decided to advance the heading without further grouting. After a suspension of nearly three months, during which time a pumping plant having a capacity of about 5 000 gal. per min., was installed in the tunnel, excavation was resumed. The first shot revealed a large water-bearing crevice to the right of the tunnel yielding about 600 gal. per min. Numerous other leaks were encountered as the heading was advanced, the total pumpage reaching a maximum of about 2 000 gal. per min. The inflow of water subsequently gradually diminished to about 1 000 gal. per min., remaining nearly constant at this figure until it was finally shut off by grouting after the lining was placed.

Better results might have been secured in this stretch if a concrete bulkhead had been used to retain the grouting pressures, although the soft and weak condition of the rock was very unfavorable for grouting. Another reason for failure to cut off the water under the conditions encountered was the difficulty of adjusting the consistency of the grout to both large and small seams. Large seams require thick grout for thorough filling, while thin grout must be used to fill the fine seams. Ordinarily, the consistency of the mix was adjusted to the large seams which have to be filled first. In order to secure the best results, grouting of the large seams should be followed by re-drilling and grouting with fine grout, to fill the narrow seams.

Grouting Water-Bearing Seam, Hudson Tunnel.—In excavating the Hudson Pressure Tunnel, 1 100 ft. below the river surface, a flow of about 500 gal. per min. was encountered in the heading near the East Shaft. A 4-in. pipe was first put in the hole from which most of the water issued, and concreted in place. The pipe was connected to the discharge line in the shaft, and the water was forced by its own pressure to a pump chamber at the side of the shaft 300 ft. above the bottom. Eight horizontal holes were drilled into the heading and plugged with pipes fitted with valves. A concrete bulkhead, 5 ft. thick, reinforced with steel rails, was then built against the face of the heading, the pipes extending through. About 180 bags of neat

cement in grout was then forced in through the pipes under pressures up to 700 lb. per sq. in. The grout was mixed in a Canniff tank and was discharged by displacing it with water pumped in by a high-pressure plunger pump.

On shooting the heading after the grout had set, there was no increase of water even after passing the seam, which was a narrow, clean-cut, vertical crack crossing the tunnel nearly at right angles and found to be well filled with grout. When the bench was excavated, exposing the seam in the bottom, it appeared to be filled with grout, but after it had been excavated about 13 ft. beyond the point of original leakage, one of the vertical bench holes struck a flow of 110 gal. per min. This flow disappeared after the next shot, and it is supposed that the hole was plugged by the displacement of the rock. The open condition of the seam in this tunnel made it easy to grout, particularly as a tight bulkhead was provided, retaining the pressure. The rock was firm and hard, and the walls of the fissure did not contain soft rock, which would have blocked the opening and prevented the free entrance of grout.

Grouting in Pressure Tunnels After Placing the Concrete Lining.

With the exception of the two instances just described, where an attempt was made to grout the rock seams at the time of driving, all grouting in the pressure tunnels was done after the concrete lining had been placed. As already suggested, the purpose of this grouting was, first, to fill with grout all void spaces between the concrete lining and the rock so that the internal hydrostatic pressures might be transmitted to the rock without rupturing the lining; and, second, to prevent or reduce the leakage out of the tunnel by forcing grout, under pressure, into the water-bearing crevices in the rock and by impregnating any porous rock surrounding the tunnel.

Placing the Concrete Lining in the Pressure Tunnels.—The concrete lining in the pressure tunnels was placed in three steps: First, the invert or bottom strip, from 4 to 6 ft. wide; second, the side-walls extending up to, or a few feet above, the horizontal diameter; and, third, the arch comprising the upper part of the section. The invert was placed continuously, with joints at the end of the day's run, which averaged about 125 ft., while the side-walls and arch were con-

creted in lengths depending on the lengths of forms used, varying from 40 ft. to a maximum of 150 ft. Steel forms were used for the side-walls and arch, the concrete being shoveled behind the forms from platforms resting upon these forms. When the concrete in the arch had nearly reached the top, longitudinal boards were set radially on either side of the top arch plate, and the concrete was banked up on a slope above the boards. Before placing the concrete in the key of the arch, the radial boards were removed and the concrete was shoveled in place in the key by end handling.

The concrete for the tunnel linings was mixed approximately in the proportions of 1:2:4 in the pressure tunnels north of New York City. The mix, however, was varied as the requirements of the work demanded. In the City Tunnel, a 1 : 1½ : 3 mix was adopted. Portland cement was used in all parts of the work. A moderately wet or plastic mix was used for the entire section, except near the top of the arch, and in the key, where a drier consistency was adopted.

Protection of Concrete from Inflowing Water During Placing.—In order to secure a dense, tight lining, it was considered essential to protect the concrete, while it was being placed and during the time of setting, from running water. The method generally adopted to accomplish this purpose was to place tin or galvanized-iron drip-pans against the rock so as to collect the water and keep it away from the concrete during placing and setting. The water from the pans ran into the tunnel through weepers, consisting of 2-in. steel or wrought-iron pipes set in the concrete while it was being placed, the pipes being extended through the steel forms by holes cut in them.

In cases of concentrated flows, holes were drilled into the rock seams and pipes were set to collect the water without the use of drip-pans, or, in some instances, both deep-seated pipes and pans were used. All flowing pipes and spaces behind the drip-pans were subsequently grouted. The drip-pans were usually held in place against the rock by nailing the edges of the pans to wooden plugs set in holes a few inches deep drilled for the purpose, and by bracing them from the forms. The space between the edges of the pan and the rock was carefully caulked with oakum or plaster of Paris, or a mixture of cement and plaster of Paris, to prevent the concrete from getting behind the pan and plugging the weeper.

In placing invert concrete, the water-bearing seams were taken care of by placing over them broken stone drains covered with cement

bags or sheets of tin, and placing grout pipes so as to have them extend down through the invert concrete and drain. Wherever necessary, the water was kept below the top of the broken stone drain by pumping through one of the grout pipes until the concrete had set sufficiently hard. In other cases, longitudinal pipe collectors were laid under the invert, carrying the water to a sump from which it was pumped. There were cases of small springs in the lower part of the side-walls, where drip-pans could not well be placed. At such places, the grout pipe was run to a blind drain made of broken stone and covered with empty cement bags and dry concrete.

The drip-pans varied in size from 1 or 2 sq. ft. to a maximum of, perhaps, 150 sq. ft. It was found to be better to use several small pans rather than one large one, in order that the grouting pressures might be applied more directly and thus be more effective in closing up the seams, and also to avoid possible rupture of the lining upon the application of the high-pressure grouting. The pans were usually fitted with two pipes, one at the bottom to drain the water and subsequently for grouting, and the other near the top to act as a vent and aid the circulation of the grout in the pan. Small pans were frequently fitted with only one pipe. A great deal of care and attention was given to the placing of the drip-pans and pipes in order to insure a tight concrete and, in addition, to make adequate provision for effective grouting subsequently.

In order to make it unnecessary to cut many holes through the forms for the pipes, it was intended originally to arrange the piping back of the forms so as to use a system of collectors with a few holes cut in the forms at regular places. It was found to be more effective, however, to use short, direct pipes for grouting purposes. There are several objections to grouting through long pipes with bends and other connections; namely, the danger of clogging the pipes, the impossibility of cleaning out the pipes or of drilling through holes already grouted for the purpose of regrouting, and the delays likely to occur in placing an arrangement of pipes with complicated details.

With the steel forms first used, holes were cut for the grout pipes and weepers only where necessary, and after repeated use, the forms were badly cut up. On some of the later work, holes were cut in the forms at standard locations. On one contract there was a threaded hole at the center of each plate, to which the pipe could be secured. This

provided an opening for each 15 sq. ft. of surface, which was adequate for ordinary conditions.

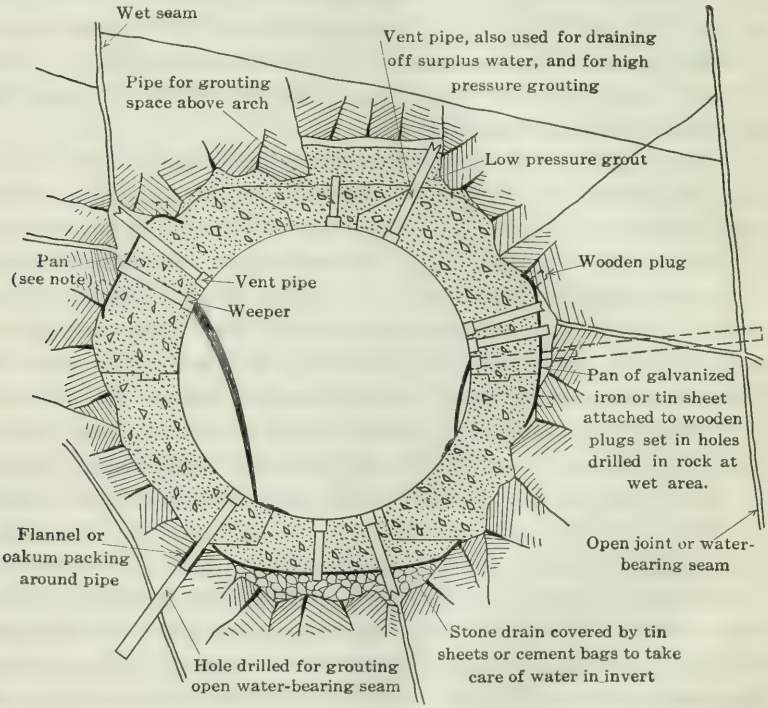
By the methods adopted for placing the concrete and protecting it from inflowing water, during setting, a dense concrete of excellent quality was obtained for the lining of the pressure tunnels. In a test made in the Wallkill Pressure Tunnel, the lining for a stretch of 180 ft. of ungrouted tunnel was subjected to a pressure of 135 lb. per sq. in. The only indications of seepage through the concrete were a few damp spots. If the concrete in the pressure tunnels had not been of good quality, it would have been extremely difficult to make the grouting operations successful.

Grout Pipes and Weepers.—In placing the concrete in the key of the arch, it is a practical impossibility to pack it tight against the rock continuously in the roof. Furthermore, in setting, it shrinks away from the rock. In order to fill the void space along the top and haunches of the arch, grout pipes were set in the arch concrete at the time it was placed. In some stretches, where the breakage in the roof was very high, the space above the normal thickness of the arch was filled with dry packing which was subsequently grouted. Where steel roof support was used, the dry packing placed between the steel lagging and the rock was also grouted through pipes set in the concrete lining.

Grout pipes in the arch were usually placed at intervals not exceeding 25 to 50 ft. along the dry stretches. In the beginning, three pipes were placed at each transverse joint in the arch, one at the center and one at either haunch, for grouting the joint. These pipes, however, were found to be of little value for the purpose and were dispensed with in later work. Additional pipes were placed at points of high breakage in the roof to serve as vents to prevent the formation of air pockets and insure the complete filling of the voids.

Both weepers and grout pipes were 2-in. steel or wrought iron, except in one of the tunnels where 1½-in. pipes were used. The 2-in. pipes are preferable as they do not clog as readily as the smaller ones. The grout pipes were carefully placed and were usually wedged between the forms and the rock, either with a spike or other spacer at the end near the rock. Each pipe was provided, at the end nearest the form, with a standard coupling to which the hose connection was made for grouting. On some parts of the work, a wad of oakum was placed in the end of the pipe nearest the rock, in order to keep out the concrete.

Upon removing the forms it was the usual practice to clean out the pipes by poking a rod into them. To prevent the displacement of the pipes during concreting, it was the practice in one of the tunnels to extend all pipes, the dry ones as well as the weepers, through the forms by means of nipples screwed into the couplings. The nipples had to be removed before the forms were struck.


Wet seam
Vent pipe, also used for draining off surplus water, and for high pressure grouting
Pipe for grouting space above arch
Low pressure grout
Pan (see note)
Vent pipe
Weeper
Wooden plug
Pan of galvanized iron or tin sheet attached to wooden plugs set in holes drilled in rock at wet area.
Flannel or oakum packing around pipe
Open joint or water-bearing seam
Stone drain covered by tin sheets or cement bags to take care of water in invert
Hole drilled for grouting open water-bearing seam

Note:

Water flowing into the tunnel from the seams in the rock is collected behind drip pans, and drains into the tunnel through 2″pipes set in the concrete while the latter is placed. The concrete is thereby protected from running water during setting. The 2″pipes are later grouted. Holes are also drilled in the rock before placing concrete, to intercept water-bearing seams. These seams are subsequently grouted through 2″pipes placed in the holes as shown.

PRESSURE TUNNEL
IN WET SEAMY ROCK
METHOD OF TAKING CARE OF INFLOWING WATER AND MAKING PROVISION FOR GROUTING WATER-BEARING SEAMS
JUNE 12, 1918

FIG. 3.

Low and High-Pressure Grouting.—Grouting in pressure tunnels was done in two stages: (1) Low-pressure and, (2), high-pressure grouting. The purpose of the low-pressure grouting was: (*a*) primarily to fill the void spaces between the lining and the rock, particularly above the arch, and, at the same time, to form a blanket over the construction joints in the arch; (*b*) to fill the voids in the dry packing where the latter was used; and (*c*) to fill any porous or honeycombed spots in the concrete.

The object of the high-pressure grouting was: (*a*) To cut off the leakage of water into the tunnel and thereby prevent the escape of water out of the tunnel by filling with grout, forced in under high pressure, the crevices and openings in the surrounding rock; and (*b*) to complete the filling of the voids between the lining and the rock, particularly in the high places in the roof, and between the drip-pans and the rock. In general, the procedure followed in grouting was first to grout the space above the arch under low pressure through the pipes set for the purpose, and then to grout, under high pressure, the vent pipes and weepers.

Low-Pressure Grouting.

Detailed Description.—The details of the low-pressure grouting differed somewhat on the various contracts, the following description being typical of the procedure as finally developed. When a stretch of tunnel was ready for grouting, the usual practice was first to grout the arch pipes, beginning at one end of the stretch and working toward the other. The pipe to be grouted was first raked out in order to clean it of any obstruction; sometimes, it was blown out with air to clear it. Plug cocks were placed on all pipes to which connections were made. A 2-in. grout-hose connection, equipped with a side vent, was attached to the pipe to be grouted, and batches of grout from a tank grouting machine were then discharged through the hose and into the pipe, by compressed air. The pressure ordinarily used for this grouting was about 75 to 90 lb. per sq. in.

Grouting was continued at each pipe to which connection was made until the pipe refused to take more grout or until the grout had traveled along the roof several hundred feet ahead, when connection was made with another pipe and the process repeated. The distance which the grout above the arch could be forced ahead of the pipe,

varied from 200 to 300 ft., and, in some cases, where there was a large continuous void space in the roof, it would travel more than 500 ft. It was the usual practice, however, where conditions were normal, to grout the space above the arch by making connections to the roof pipes every 125 to 200 ft.

In charging the grout tank the cement, sand, and water in proper proportions were placed in the tank, and the air was allowed to boil up through the batch to agitate it, while, at the same time, the batch was generally stirred by a hand paddle, as the air alone was insufficient to mix the materials thoroughly.

As the batches of grout were forced into the pipe and the grout flowed ahead, water in large streams issued from the adjacent roof pipes; this was the excess water from the grout, as well as the ground-water leakage which had collected above the arch. This water was pushed ahead of the grout, and as soon as the pipes showed a flow of grout they were plugged either by driving tapering pine plugs into them or, as in the case of the vent or other pipes to which connection was to be made for high-pressure grouting, they were closed by means of the plug cocks at the ends. In some cases, the holes for the connections were plugged by driving a wad of oakum back into the far end of the pipe. At the end of the shift, or several hours after grouting, when the grout had partly set, the oakum was removed with an auger. Any pipe which had filled up with grout that had set, was drilled out for the subsequent grouting.

During low-pressure grouting, the high vent pipes were kept open to drain off the excess water, to permit the escape of air, and to prevent the accumulation of ground-water pressure. During the grouting, the transverse joints in the arch and any porous spots in the concrete generally showed a seepage of water first, then very thin grout appeared, after which, the leaks usually stopped in a short time, and silted up. In some cases, the leakage of grout and water through the joints was stopped by driving thin wooden wedges into the joints, and leakage through the porous concrete was stopped by bracing and wedging a board covered with a mat of oakum against the porous spot. The longitudinal joints on either side of the key at the top of the arch also leaked in a few stretches, due principally to porous concrete in the vicinity of the joints.

The weepers from drains and drip-pans, and the deep-seated pipes were also kept open during low-pressure grouting. The matter of proper venting and free drainage of the ground-water during low-pressure grouting is of the greatest importance in securing good results, because it is essential not to permit the building up of a ground-water head on the fresh grout since the object is to form an impervious blanket over the top of the arch and thereby prevent leakage of water through the unset grout and the formation of channels through the transverse and longitudinal construction joints or through any porous spots in the concrete.

Grouting Dry-Packed Spaces Above the Arch.—When grouting dry packing in large void spaces above the arch, hose connections were first made to the lowest pipes, and as the voids in the dry packing were filled and the water was drained off, connections were made to the higher pipes. Cases occurred where the vent pipes extended from 15 to 20 ft. in the dry packing above the arch, and, at these places, grouting was continued until the vent pipes showed a flow of grout, indicating that the dry packing was thoroughly filled. Where water is present above the arch, it is important that the grout should displace the water from below and not be permitted to mingle with it. Samples cut from the grout above the arch showed the grout to be exceedingly dense and impervious, filling the space without shrinkage and often forming a close bond with the surrounding rock.

Proportions Used for Low-Pressure Grouting.—A batch of grout consisted of one bag of cement weighing about 95 lb., one bag of sand weighing generally about 80 to 90 lb., and from 6 to 8 gal. of water, the total making a batch of about 2.0 to 2.2 cu. ft. of liquid grout measured in the tank. The cement was the ordinary commercial brand used for the concrete lining. The sand was very fine, screened when necessary to meet the requirements of the specifications, namely, that 100% pass a sieve having 64 openings per sq. in., 45% pass a sieve having 1 600 openings per sq. in., the wires of the sieves being, respectively, 0.035 and 0.013 in. in diameter. In the work north of New York City, glacial sand was used, while for the City Tunnel white quartz beach sand from the south side of Long Island was obtained.

The quantity of grout placed depended on the excess breakage in the roof and the extent of the dry packed spaces. In the City Tunnel, in unsupported stretches, where no dry packing was used in the roof

the quantity of grout placed under low pressure varied from 4 to 9 cu. ft. per lin. ft. of tunnel.

Progress in Low-Pressure Grouting.—The progress in low-pressure grouting varied, usually from about 600 to 1 000 ft. of tunnel a week, working three shifts a day. Many factors influenced the speed, such as the length of haul, the delivery of materials, the proficiency of the gang in loading and unloading the cars, etc., and, of course, the volume of space to be filled above the arch. Usually, in normal stretches of tunnel, not more than one or two connections were made in 24 hours. The sand and cement were hauled through the tunnel to the grouting plant on flat cars holding from 20 to 30 bags. The sand was brought to the work in bulk, but was bagged by hand at the top of the shaft before it was sent underground. Tests were made of handling the sand in bulk in the tunnel, but it was soon found that a package of about the same size and weight as a bag of cement was the most convenient form for rapid handling.

The force employed in the tunnel for low-pressure grouting was generally one foreman and seven or eight men, two charging the grout tanks, one stirring the batch, one operating the valves on the grout tank, and three or four handling the hose and making the connections. On the surface, men were employed to bag the sand and load the cars on to the cages.

Precautions to Be Observed in Placing Low-Pressure Grout.—Low-pressure grout should be injected gently under as low a pressure as possible to avoid churning of the mix and separation of the ingredients, and only a sufficient quantity of water should be used to produce a freely flowing mixture that will not clog the hose. The air pressure should not be higher than necessary to force the grout through the hose and into place. A reducing valve was used on some parts of the work to cut down the line pressure from 90 to 40 and 30 lb. per sq. in. An excess of air, especially when injecting grout under water, is likely to agitate and churn the mix, causing separation and stratification of the sand and cement, with the formation of a soft deposit of laitance at the top of the grout in contact with the rock.

To avoid the after blast of air when grouting a space filled with water, it was the usual practice to close the discharge valve before the hose "kicked", the latter requiring careful watching. A further precaution used to prevent the separation of the mix where a large

space filled with water was grouted, was to keep the grouting connection from 2 to 3 ft. below the top of the grout.

The importance of venting the high pipes and allowing all the weepers to run freely during low-pressure grouting has been referred to. It sometimes happens that low-pressure grout enters the drip-pans or drains, but in such cases the pipes should not be kept closed to allow the accumulation of ground-water pressures.

In stretches showing general seepage in the roof of the tunnel, taken care of by drip-pans and pipes, it was sometimes found that the panning was not entirely successful in preventing the water from mingling with the grout. At first, it was thought that these wet stretches should not be grouted with the drier stretches on either side, and they were skipped on the first trip, but it was found that better results could be secured by grouting the wet stretches in the roof with the rest, as the water in the wet stretches did not accumulate pressure, but would travel over the top of the dense unset grout to open vent pipes, sometimes a long distance away, instead of breaking through the grout blanket at a point of near-by weakness.

High-Pressure Grouting.

The purpose of the high-pressure grouting, as already stated, was to complete the filling of the voids between the lining and the rocks, and, more particularly, to cut off the flow of water into the tunnel by forcing grout into the water-bearing seams and closing up all openings in the surrounding rock, in order to prevent or reduce the leakage out of the tunnel when the latter is in service.

Detailed Description.—It was the general practice first to grout the high vent pipes in the roof, in order to complete the filling of the space above the arch, and then to grout the pipes from the drip-pans and the deep-seated pipes leading to the rock seams. Some of the vent pipes in the roof showed a flow of water in stretches originally dry. The grouting of one vent pipe would often stop the flow of water from another, indicating a flow of grout through the open seams in the rock. In other cases there would be an increase in the flow of water from a weeper due to forcing water through the rock seams to an open vent. As a rule, very little grout could be injected into the vent pipes.

After grouting the arch pipes, the "wet" pipes, or weepers, were next grouted. In starting operations, air at 100-lb. pressure was

blown into the pipe to expel the water from the drip-pans or seams. This was sometimes followed by a charge of water to clear the pipe just prior to grouting. The charge of water was often applied as a test to give an idea of the circulation of grout to be expected. If the charge of water failed to "go", the pipe would take very little grout. Another indication of the size and extent of the rock seams that could be reached from a connection was the showing of air at another open pipe or the refusal of the pipe to take air when high air pressure was applied.

The usual practice was to grout the weepers with a mix of 1 bag of cement and 5 to 8 gal. of water. When the pipe would take no more grout, the high pressure, about 300 lb. per sq. in., was kept on it for from 5 to 10 min., which had the effect of squeezing the water out of the grout and packing the cement solidly in the pipe. Frequently, if this procedure did not completely fill the pipe, it was drained and plugged with a thick batch of grout. Temporary plugging of a seam during grouting was often overcome by forcing air into the seam under high pressure.

It was found difficult to shut off small leaks through pipes set in the lower portion of the side-walls where no vents were provided. After a pressure of 300 lb. per sq. in. had been maintained on these holes for several minutes, it was often found that no grout had been forced in the pipe and the leak still continued. Such pipes were successfully closed, even in the case of those which flowed several gallons per minute, by driving a tight-fitting wooden plug into the extreme end of the pipe and then ramming with dry cement.

It was the practice on some parts of the work to complete the high-pressure grouting on all the key pipes in the arch in a stretch of tunnel, after which the tunnel was gone over a third time when the flowing pipes were grouted.

Air Pressures.—The pressure used for high-pressure grouting varied, as a rule, between 250 and 350 lb. per sq. in. This was the pressure at the grouting tank. It had to be sufficiently high to overcome the ground-water pressure and, in addition, to force the grout into the seams in the surrounding rock. In the case of very fine seams, the frictional resistance is so great that it is doubtful if much grout is injected into them even under high tank pressures.

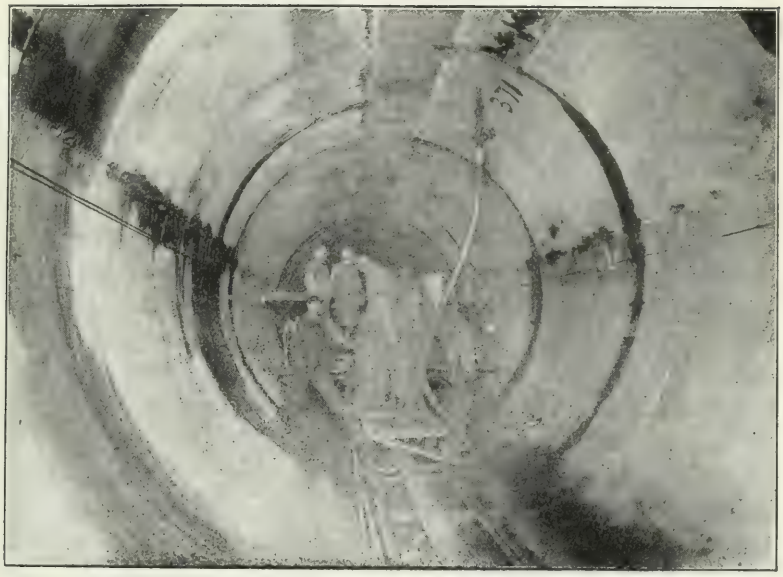


FIG. 4.—GROUTING EQUIPMENT IN OPERATION IN CITY TUNNEL.

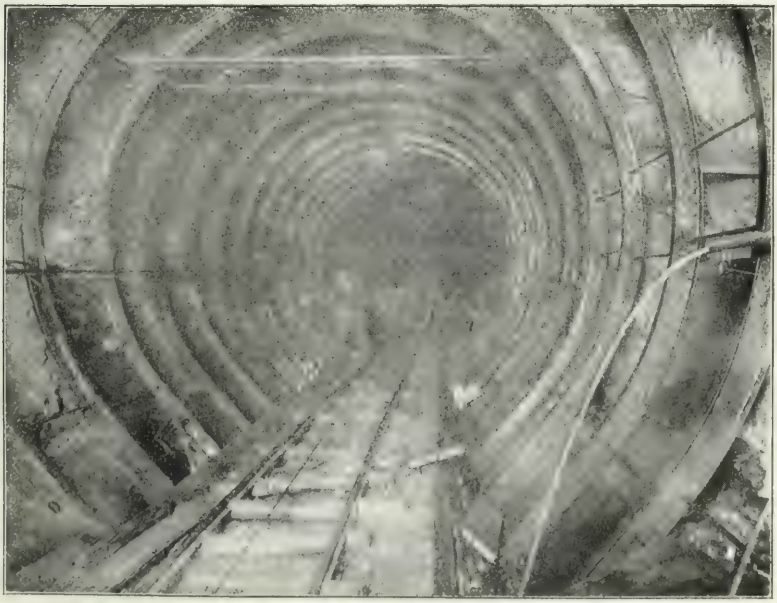


FIG. 5.—STEEL SHELL FOR TAKING CARE OF WATER, RONDOUT PRESSURE TUNNEL. FLOW OF 2 000 GAL. PER MIN. ENCOUNTERED IN STRETCH OF 350 FT.

Gauge measurements of the rock-water pressure were often taken before the grouting of a flowing pipe was started. The gauge readings, however, gave no clue as to the quantity of grout the pipes would take. The gauge pressures showed that the rock-water level at the pressure tunnels had dropped considerably below the original ground-water level, the gauges registering pressures usually under 100 lb. per sq. in., so that the back pressure was not generally high and the grout was forced in under high net pressures. In many cases, the entire ground-water head was restored a few weeks after the completion of the high-pressure grouting. The grouting of a wet seam in the Hudson River Tunnel has already been referred to. At this location, the ground-water pressure before grouting was 475 lb. per sq. in., and the grouting was done under pressures running up to a maximum of about 700 lb. per sq. in.

Mix and Consistency.—To insure the flow of grout into the fine openings, neat cement was used in high-pressure grouting. The consistency of the mix varied, depending on the conditions. Ordinarily, 1 bag of cement and 5 to 8 gal. of water were used. Where grout could be forced into the rock seams it was mixed very thin—as thin as $\frac{1}{4}$ bag of cement to 30 gal. of water. If this mix was taken freely, it was regarded as an indication that the grout was traveling some distance from the tunnel lining and, therefore, was not of much value. The mix was accordingly thickened gradually by increasing the quantity of cement in the batches until the pipe began to refuse grout, when thin grout was again injected, the pipe being finally plugged with a batch of thick grout under 300-lb. pressure.

Cement.—It was originally contemplated to use very finely ground cement for grouting in order that the grout might be carried readily into the finest seams. However, cement of ordinary fineness of grinding was used, as it was realized that, in time, the openings outside the tunnel lining and through the body of the concrete would silt up. Experiments show that the finer the grinding, the greater the loss in the strength of the grout on account of the more thorough hydration of the cement. The coarse particles impart strength to the cement, and, for that reason, very fine grinding may not be desirable. Some tests were made under severe conditions, using quick-setting Lias lime, but the results were not satisfactory, as the material did not harden properly and the water was not cut off.

Precautions to Be Observed in High-Pressure Grouting.—A very important factor in successful grouting is the control of the consistency of the mix. For large openings, thick consistencies should be used, while for the very finest openings the best results are obtained with thin grout. The thick mixes, however, must carry enough water to prevent undue clogging. No coarse particle or lumps of cement must be allowed to go into the tank. The cement is sometimes affected by the dampness of the tunnels and should be screened when necessary. Grouting of a hole must be continued without an interruption long enough to clog the hose or permit an initial set of the grout.

Special Features in Connection with Grouting.

Time Required for Grouting and Quantity of Grout Placed.—The time required for grouting the pressure tunnels varied, depending on the method and equipment used, the length of haul, the extent of the space grouted, and the quantity and distribution of inward leakage. Table 1 and the following data give the progress, force employed, power consumption, and other facts relating to the grouting in the two miles of tunnel included in Section 8 of the City Tunnel, in the lower part of New York City. This portion of the tunnel was comparatively dry, there being no large, concentrated flows within the two miles. The air used for grouting was supplied by a compressor at the top of one of the shafts. The plant and equipment used for grouting on this part of the work are shown in Fig. 6.

TABLE 1.—Data on Pressure Tunnel Grouting, Contract 67, Section 8.

Finished Diameter of Tunnel, 11 and 12 Ft.

	Low-pressure grouting: One grouting outfit.	High-pressure grouting: One grouting outfit.
Linear feet of tunnel grouted	10 313	10 313
Connections to grout pipes	170	1 683
Eight-hour shifts worked	72	116
Batches of grout placed	23 298	2 888
Quantity of liquid grout placed, in cubic yards	1 761	153
Quantity of grout per linear foot of tunnel, in cubic yards	0.17	0.02
Progress per week, in linear feet of tunnel	2 690	1 457

The total force employed in the tunnel and on the surface, averaged 42 men on each shift during the low-pressure grouting and 34 men per shift on high-pressure grouting, for two grouting outfits.

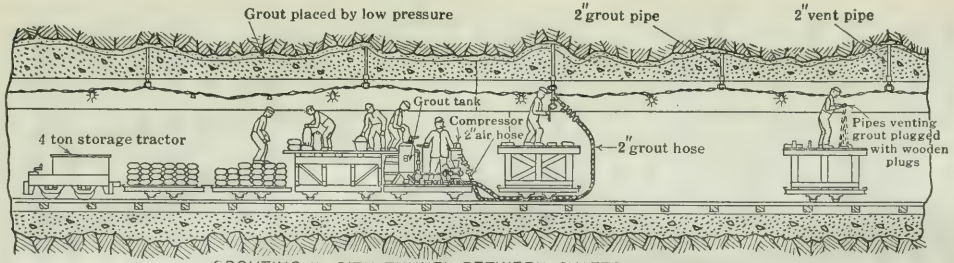
The electrical power consumed by the compressor, hoist, lighting, pumps, and tractors aggregated approximately 7 800 kw-hr. and 5 300 kw-hr. per 1 000 ft. of tunnel on low and high-pressure grouting, respectively, covering two grouting outfits.

The cost to the City, of grouting, based on contract prices, averaged $5 per lin. ft. of tunnel, on Contract 67.

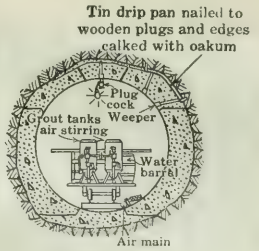
Quantity of Grout Taken by Flowing Pipes.—The quantity of grout injected into the pipes varied, depending on the flow of water, the size of the openings in the rock, the back pressure, and other conditions. The quantity of water issuing from a flowing pipe gave some indication of the quantity of grout the pipe would take, but it was not an absolute index. An idea of the quantity of grout taken by flowing pipes is furnished by the experience in grouting seams in limestone in the Rondout Siphon, where the tunnel lining was ruptured on account of movement in the surrounding rock when the tunnel was first filled with water for the purpose of testing it. Holes were drilled through the lining into the rock, and the seams were grouted. A very thin grout was used—about 0.8 lb. of cement to 1 gal. of water.

In the case of 29 pipes flowing less than 1 gal. per min., only 6 took more than 1 bag of cement, and the maximum in any one pipe was 2.5 bags. Of 24 pipes leaking from 1 to 5 gal. per min., the maximum quantity of grout injected into any one pipe was 24 bags of cement, the minimum, 0.25 bag, and the average, 4.3 bags. Of 12 pipes leaking from 5 to 10 gal. per min., the maximum quantity injected into any pipe was 7.5 bags, the minimum 1 bag, and the average 3.2 bags. Of 8 pipes leaking more than 10 gal. per min., the maximum quantity injected into any one pipe was 198 bags, the minimum 5 bags, and the average 62 bags.

Cut-off Walls.—The method of grouting originally contemplated called for the construction of masonry cut-off walls to be built over the tunnel arch tight against the roof, at intervals of about 50 ft. The purpose of these walls was to confine the flow of grout and to grout the space above the tunnel arch in short stretches, the pressure in the stretch grouted being raised gradually until the maximum required pressure of 300 lb. per sq. in. was reached. It was expected that the high pressures would force the grout into the seams in the surrounding rock. The cut-off walls were to be constructed so that they would retain grout under the maximum pressure.



GROUTING IN CITY TUNNEL BETWEEN SHAFTS 19 AND 21


Tin drip pan nailed to wooden plugs and edges calked with oakum
Plug cock
Grout tanks Weeper air stirring
Water barrel
Air main

SECTION

GENERAL DESCRIPTION

The operation of grouting the tunnel was divided into two distinct stages: Low-pressure grouting and high-pressure grouting. The object of the former was primarily to fill the space at the top of the arch left by the shrinking of the concrete away from the rock and to fill openings at joints and honeycombed places. The object of the latter was to fill open seams in the rock surrounding the tunnel and all pipes and pans placed to control water seeping into the tunnel.

LOW-PRESSURE GROUTING

Each unit consisted of a grouting car equipped with two grout tanks, one cement car, two platforms on wheels, six flat cars, two electric storage tractors and three shifts of men. Air at a pressure of 100 lb. per sq. in. at the shaft compressor was used in the machines at a pressure of 95 lb.

Only pipes located in the key portion of the arch were connected to in low-pressure grouting.

The grout hose was connected to pipes situated at points where the rock roof was low; pipes at high points acted as vents. As grout flowed along the top of the arch, first water appeared through joints between successive days' work and from grout pipes embedded in the arch; this water was followed by grout which became thicker and thicker and finally clogged all leaking joints. All pipes running grout were stopped with wooden plugs and the valves on key pipes were closed.

In low-pressure work the hose was connected directly to valves on key pipes instead of on the Y-branches shown. Grout was forced ahead 100 to 300 ft. before the machine was moved to the end of the grouted portion. The standard low pressure batch was 1 bag of cement, 95 lb. of sand and about 7 gal. of water. This batch made 2.04 cu. ft. of liquid grout or 1.92 cu. ft. of set grout.

HIGH-PRESSURE GROUTING

Twenty-four hours after any section of tunnel had been grouted by low pressure, all pipes which were to be connected to were cleaned out to rock to allow effective treatment by high-pressure grouting.

Each unit consisted of one grouting car equipped with one grout tank and one booster compressor, one cement car, two platforms on wheels, three flat cars and two electric storage tractors and generally one shift of men. Air was delivered from the shaft compressor to the booster compressor, which raised the 95 lb. pressure to 300 lb.

High-pressure grouting was accomplished generally in one trip over the section of tunnel. The grout hose was connected to the pipes by a Y-branch, as shown, to allow a rod to be forced in for the purpose of freeing a passage for the grout which often became jammed in the pipes by the high pressure.

All grout pipes that were connected to had grout forced into them at 300 lb. pressure, which was maintained for 5 min. The standard high-pressure batch was 1 bag of cement and 5.5 gal. of water, which made 1.24 cu. ft. of liquid grout or 1.2 cu. ft. of set grout.

FIG. 6.—METHOD OF GROUTING IN PRESSURE TUNNEL

In the beginning efforts were made to build these cut-off walls, but, after some experience, it was found to be impracticable to make them tight against the high pressures. Aside from this consideration, there is serious objection in the case of wet stretches to dividing the arch into short sections and at once applying high-pressure grout. If the ground-water is confined, the head builds up, and on the application of high-pressure air to the liquid grout over the arch, the ground-water will force an outlet into the tunnel through the transverse construction joints in the arch. Efforts to stop such leaks by drilling holes in the concrete lining and rock and subsequent grouting were generally unsuccessful.

After the originally contemplated method of building the cut-off walls was abandoned, an attempt was made to build grout cut-off walls, and to grout in three stages, substantially as follows: A connection was made to a pipe set at a low spot in the roof profile, and grout was forced in under low pressure, generally from 30 to 40 lb., until it flowed from an adjacent pipe the upper end of which was at a higher elevation. Another low spot in the roof, about 75 to 100 ft. distant, was then grouted, and the process was continued throughout a stretch of tunnel. If any weepers showed grout during this stage, they were plugged, and the holes were reopened about 12 hours later by cleaning out the pipes with a rod. Subsequently, connection was made at these holes for high-pressure grouting.

After the grout cut-off walls had set up, the second stage consisted in grouting the sections between the cut-off walls, starting with a pressure of 100 lb. and raising it to 300 lb. per sq. in. The high vent pipes were then opened and the water which had accumulated was allowed to run off. After a stretch of tunnel had been grouted in this manner, all the valves at the ends of the pipes were opened, all plugs removed, and all pipes which were open and flowed water, were grouted under a pressure of 300 lb. After this method of grouting had been tried, it was found that more than 70% of the total quantity of grout placed was used for forming the cut-off walls. There was therefore little or no advantage in this scheme, and it was abandoned, and the method of grouting the entire space above the arch in one operation without the use of cut-off walls, as already described, was adopted.

A fair degree of success was attained in the use of cut-offs in one stretch in the Rondout Tunnel, where the shrinkage space between the

concrete in the arch and the rock in the roof at the end of each section of arch was packed tight with mortar. The method of grouting was to fill the space above a section of the arch with 1:1 grout at a pressure of about 50 lb. per sq. in. and, after this space was filled, to continue forcing neat cement grout through the same connection until the hole refused to take grout at 300-lb. pressure. High vent pipes in the same section of arch were then grouted under a pressure of 300 lb. per sq. in. The conditions in the stretch where this method was used were very favorable, the arch being practically dry and the breakage in the roof regular. Not all the cut-offs proved to be tight, however, and the grout was not confined in all cases. An interesting test, showing the reduction of tank pressure, was made during the grouting in this stretch. With an air pressure of 300 lb. per sq. in. at the grouting tank, a gauge at a distance of 15 ft. from the pipe connected to, registered a pressure of 160 lb. per sq. in., the 1:1 grout over the top of the arch having been in place less than 4 hours.

In stretches of tunnel where dry packing was used above the arch continuously for several hundred feet, cut-off walls were built at intervals of from 75 to 100 ft. to avoid the flow of grout for very long distances, and to prevent the separation of the sand and the cement. Brick cut-off walls were built in connection with the grouting of the wet stretch on the incline north of Shaft 4, Rondout Tunnel. The grouting of this stretch is described in detail later.

Drip-Pans vs. Deep-Seated Pipes.—There may be a difference of opinion as to the best method of treatment in taking care of incoming water and subsequent grouting where the leakage into the tunnel extends over a considerable area. Although in numerous cases deep holes were drilled into the water-bearing seams to collect the water, it was not the general practice to drill an extensive number of such holes where large wet areas were encountered. The use of drip-pans, whereby all the open seams are exposed directly to the action of the grout, is believed to be the best method of treatment in such cases. In either method, penetration of the grout is probably very slight in the case of small openings in the rock.

In two cases where excessively wet stretches of tunnel were encountered, namely, in Rondout Tunnel, north of Shaft 4, and in the City Aqueduct Tunnel, near Shaft 24, light steel shells consisting of ribs and plates were used to protect the concrete during placing. (See Fig. 5.)

The space between the shell and the rock was dry packed and subsequently grouted. In addition, holes were drilled directly into the seams and grouted. Very successful results were obtained by the methods adopted in these two stretches. There were other places where deep holes were drilled in wet ground and grout forced in the rock with very good results. Where conditions are favorable, that is, in porous and broken-up rock with open and distinct seams, permitting the free circulation of the grout, deep-seated pipes are undoubtedly very effective in grouting the rock. A sufficient number of deep holes should be drilled, however, so as to intersect as many seams as possible.

For a short distance south of Shaft 14 in the City Tunnel, practically the whole face of the rock was dripping water. To avoid covering the entire area with drip-pans, parallel strips of pans, about 2 ft. wide and a few feet apart, were placed around the upper half of the tunnel. It was thought that the water would find less resistance in traveling through a few feet of fissured rock than through the newly placed concrete lining. This proved to be the case, as all the water came through the grout pipes connected with the pans. Through these same pipes the tunnel was grouted very successfully.

To test the efficacy of the grouting of the rock by means of deep-seated pipes, test holes were drilled through the concrete lining in the Rondout Tunnel and into the rock in the very wet stretch. With one exception, the holes showed a slight seepage, indicating that the grout had filled the water-bearing seams adjacent to the tunnel. In one hole, however, drilled about 8 ft. into the rock in the invert, a flow of 71 gal. per min. was struck, although 133 bags of cement had been forced into a hole about 5 ft. away during the original high-pressure grouting. This test shows that the character of the rock, the nature and size of the openings, and other conditions must be carefully studied in locating the holes for grouting. Another requisite for success in grouting the deep-seated pipes is the use of thin grout. It is also important to inject the grout continuously, so that the ground-water pressure will not force it back into the pipe or hole.

Deep holes drilled into very fine seams will ordinarily clog the seam with drill chips, making it impossible to force much grout into such seams. Experience showed that not more than one pipe in ten would permit the grouting of fine seams. Deep holes are preferable to shallow holes, as the former are more apt to intersect water-bearing seams and

take more grout. If deep-seated pipes are used along a large water-bearing seam, they should be placed generally not more than 15 ft. apart; the seams where exposed should be raked out to clear them of loose chips of rock, and the edges should be caulked with oakum or covered by a drip-pan before placing the concrete. The difficulty of adjusting the consistency of the grout, where large and small seams occur, has been referred to. Wide seams require thick grout, while fine seams are at once blocked without filling, unless the thinnest grout is used.

Time of Grouting.—The question of the time to be allowed between the placing of the concrete lining and the application of high-pressure grout was given careful consideration, particularly where the tank grouting pressure might be transmitted over a considerable area. It was stipulated in the specifications that grouting under pressure would not generally be ordered in any place until three months after placing the concrete lining at that place. This rule was generally followed, although there were cases where grouting under 300 lb. pressure was done within a month after the placing of the concrete without injury to the lining. There were two or three cases where the lining ruptured at the time of grouting, but the breaks were not due to the lack of age of the concrete. No case of cracking occurred in the top of the arch.

Other considerations controlling the time of grouting are the shrinkage and the imperviousness of the concrete. Tests show that, at the age of 1 month, concrete has developed 80% of its 3 months' strength and has acquired the greater part of its imperviousness. Shrinkage also appears to be complete in from 3 to 4 weeks. It would seem, therefore, that grouting under ordinary conditions can safely be done one month after the placing of the concrete lining.

Cracks in the Concrete Lining.—Reference has been made to the opening of the transverse joints between adjacent sections of the arch and side-walls, due to the contraction of the concrete. Irregular transverse shrinkage cracks also developed between the construction joints, even in the 30 and 40-ft. lengths of arch and side-wall. The cracks were mostly in the arch, but were quite common also in the side-wall. This cracking is due to the contraction of the concrete in cooling from a temperature of 100 to 160° Fahr. during setting, to the normal temperature in the tunnel which in summer was not much above 60° and in the winter sometimes fell below 32° Fahr. near a shaft.

The frequency of the intermediate cracks in the lining was affected by the time of year the concrete was placed. In the City Tunnel, for example, it was found that there were more cracks in the concrete placed during the warm summer months than in that placed during the winter. The richness of the concrete also may have influenced the cracking. There was some inward leakage through the intermediate cracks in the lining after the grouting in the tunnel was completed and the ground-water pressure had accumulated. Most of these leaks, however, silted up after some interval.

There were only two cases where the lining cracked during the high-pressure grouting. In the Moodna Tunnel, several breaks developed in one stretch in the side-wall concrete, the cracks extending in places into the invert concrete. Examination showed that the breaks in this stretch resulted from defective work in connection with the placing of the concrete lining, the defects being due principally to inadequate measures for taking care of the inflowing water which accumulated behind the forms, resulting in the formation of laitance and weakening of the concrete. In grouting the Hudson Tunnel, several cracks developed in the lining under a grouting pressure of 600 lb. per sq. in. On cutting out the concrete at one of the cracks, where a large drip-pan had been used, it was found that a soft deposit had formed between the pan and the rock, so that it had been impossible to fill this space effectively with grout.

Leakage Through Construction Joints.—In wet stretches some difficulty was experienced in cutting off the leakage of water into the tunnel through the transverse joints in the arch. These joints opened up, due to the contraction of the concrete caused by the drop in temperature from the time of setting to the time of grouting. Holes drilled into the joints through the concrete and grouted proved ineffective, as the running water would wash out the grout.

Various schemes were tried to make the transverse joints tight, among which were the following: Grouting grooves were formed at the end of each arch section, either by embedding a slit rubber hose in the concrete or by nailing a strip of sheet iron over a recess left in it at the joint. These grooves were grouted through pipes set in the concrete. The results were not entirely successful in cutting off the leakage. Better results were obtained by the use of steel plates embedded in the arch joints, the purpose of the plates being to provide a

water-stop as well as to retain the grout. These schemes were used only to a very limited extent at the beginning of the work, and, later, were abandoned.

Very good results were obtained in closing wet transverse joints by hand-caulking them. Grooves about 1½ in. deep were cut along the joint and ¼-in. lead wire was hammered in. This method proved quite effective in making the joints tight.

The longitudinal joints between the invert and side-walls were found to be tight. At the joint between the side-walls and the arch some leakage occurred in the wet places. There was also some inward leakage in places at the longitudinal joints in the arch. The greater part of this leakage was due to porous concrete in the vicinity of the joint, where a relatively dry concrete had to be used, and part may have been the result of a slight opening of the joints due to the lateral contraction of the concrete.

Inspection of Grout in Place.—In order to see how effective the grout was in filling void spaces back of the concrete lining, test holes were cut in the arch concrete in several places in the pressure tunnels. Blocks of concrete about 1 ft. square were cut out in the arch, exposing the rock in the roof. It was found that the space between the top of the arch concrete and the rock was well filled with grout, and tests showed the grout filling to be dense and hard. As a somewhat dry consistency is used for the concrete in the top of the arch, this concrete when placed is likely to be porous in places; in the concrete cut out of the test holes, the grout was found to fill the voids in this concrete, and the entire block was a dense and uniform mass that was in solid contact with the rock. Test holes were also cut from the lining in the Rondout Pressure Tunnel where grout was used to fill the shrinkage space at the top of the concrete lining placed inside of a steel reinforcing shell. Here, also, excellent filling was found between the concrete and the steel.

Several opportunities were also afforded to observe the results of the grouting during the progress of the work. In grouting the wet seam in the Hudson River Tunnel near the East Shaft, already referred to, the seam, as stated, was found to be well filled with grout when exposed by the excavation. In sinking the wet shaft, known as Shaft 4, at the Rondout Pressure Tunnel, grouting was resorted to, as the shaft was deepened to cut off the inward leakage, and the sub-

sequent excavation through the grouted zone showed the seams in the rock well filled with grout. The grouting in this shaft is described subsequently in more detail.

In Shafts 22 and 24 of the City Tunnel, where the wet portions were grouted as the shafts were sunk, subsequent excavation showed that all but the very fine seams were completely filled with grout.

Reduction of Inward Leakage After Grouting.—An idea of the effectiveness of the concrete lining and the grouting in cutting off leakage into the pressure tunnels is given by the data in Table 2.

TABLE 2.—LEAKAGE INTO PRESSURE TUNNELS.

Pressure Tunnel.	Length, in feet.	Leakage into Tunnel, in Gallons per Minute.	
		Before grouting.	After grouting.
Rondout, wet stretch north of Shaft 4	350	850	3
Rondout (exclusive of above wet stretch)	23 650	230	19
Wallkill	23 390	506	75
Moodna	24 100	496	85
City, wet stretch near Shaft 23	90	825	8
City (exclusive of above wet stretch)	93 810	973	127

Hydrostatic Tests of Pressure Tunnels.—After the pressure tunnels and shafts were completed, they were filled with water in order to test their tightness. The tests in some cases were unusually severe, as the full ground-water head had not been restored at the time the test was made. At the Rondout Pressure Tunnel, the maximum unbalanced head at the beginning of the first hydrostatic test was approximately 500 ft. After the tests, the tunnels were unwatered for inspection and repairs, and the results were found to be very satisfactory, the tunnels proving remarkably tight.

Only in four places, in the 35 miles of pressure tunnel, aggregating in length about 2 500 ft., did any weakness develop, due to the yielding of the surrounding rock, causing rupture of the lining and outward leakage and making subsequent repairs necessary. In these cases, it would have been impossible to reveal fully in advance the geological conditions, particularly where the rock appeared to be sound when the tunnel was excavated, and it is believed that no amount of grouting prior to the application of the test pressure would have been adequate to consolidate the ground sufficiently to prevent the yielding of the rock. The cost of testing the tunnels and making the necessary

repairs was very much less than it would have been to make the tunnels absolutely secure by the use of steel reinforcement in the concrete lining for their entire length.

Equipment for Grouting.

Air-Stirring Grout Tanks.—Tank grouting machines of the Canniff type were used almost exclusively for grouting. These machines are very simple in construction and operation, consisting essentially of a tight cylinder provided with suitable pipe connections for introducing compressed air and for mixing and ejecting the grout. The tanks are very compact, weigh about 500 lb. when empty, and have no moving parts to wear out and be replaced. They are, therefore, particularly well suited for tunnel work. Both thick and thin mixtures can be used in them, and they are equally well adapted for high and low-pressure work. They are very efficient where large quantities of grout have to be placed.

A slight disadvantage of the Canniff tank is found when grouting fine seams under high pressure, or placing small quantities of 1:1 grout. As the process is necessarily slow there is a tendency for the cement, but more particularly the sand, to settle in the bottom of the tank and clog the openings. The use of very thin grout usually overcomes the difficulty.

As a rule, two grouting tanks were operated simultaneously in grouting the tunnel, one being charged while the other was discharging. For high-pressure work, however, it was the usual practice to use only one tank. The two tanks were mounted either alone on a low flat car, or at one end of a platform car which was large enough to hold, in addition, from 20 to 30 bags of sand and cement. In some cases, an upper platform was also provided for materials. The materials were hauled through the tunnel by mules or motor trucks. The water for mixing was measured either by pails, or automatically by a tank placed at the side or above the grout tanks. High-pressure armored hose about 25 ft. long was used for grouting purposes. The plant used for grouting on the work at the lower end of the City Tunnel is shown in Fig. 6.

Operation of Grout Tanks.—The usual method of operation during low-pressure grouting, was first to put into the tank the required quantity of water, then to let the air into the tank at the bottom.

which agitated the water, and then the cement and sand were put into the tank and stirred by hand with a wooden paddle, as it was found that the air was too slow for mixing and was also wasteful. When thoroughly mixed, the door at the top of the tank was closed, the air was then admitted at the top, and the liquid grout forced out through the grout hose into the pipe connected to. The air was then turned off quickly.

Unless care is taken to close the air discharge valve the instant the last of the grout leaves the tank, an after-blast of air follows the grout, resulting in stirring it up and causing separation of the materials, and also tending to form air pockets in the space grouted. The men operating the tanks, however, soon became skilled in controlling the discharge. As an added precaution, the air-pressure gauge on the grout tank was watched closely to note the fall of pressure as the grout discharged, and the emptying of the tank was also indicated by the "kick" of the grout hose when the charge left it.

In some cases the following scheme was tried. The air valve was closed as the batch of grout started to leave the tank, permitting the air in the tank to expand and force the grout through the hose. This method, however, was slow and was not generally used. Where the grout displaced water, the ejection of the grout by expanding the air in the tank was quite effective in cutting down the after-blast of air. The tanks could be operated very rapidly. As much as 1 500 batches of 1 : 1 grout, equivalent to about 115 cu. yd., were placed in 24 hours, using two tanks. This process involved the handling of about 135 tons of cement and sand in the 24 hours.

High-Pressure Air Compressors.—The air for operating the grout tanks was obtained by connecting to the air line laid in the tunnel, the air pressure in the line being usually from 80 to 100 lb. In order to provide the pressures required for high-pressure grouting, small Westinghouse direct-acting single-stage air compressors, known as "boosters", were generally used. These compressors, mounted on the same car with the grout tanks, took suction from the main air line and delivered air in the tanks at a pressure of 300 lb. per sq. in. The compressors weigh about 500 lb. They are provided with air gauges registering up to 450 lb. per sq. in. A spring relief valve, capable of being set at any pressure up to 400 lb., is placed on the main discharge. Each branch of the 2-way discharge is supplied with a stop valve, a

check valve, and air emergency stop valve, so that it may be closed automatically by a sudden release of pressure due to the bursting of hose or any other reason.

Independent Air-Generating Plant in Tunnel.—The grouting in the tunnel necessitated the maintenance and operation of the large compressor plant used for tunnel driving and the re-laying of the air pipe line in the tunnel, which had been taken up during the placing of the lining. In order to avoid the use of the expensive main air plant some of the contractors used small independent portable compressors in the tunnel, driven by gasoline or electricity, and mounted on cars traveling along with the other equipment. These compressors were used both for the low and high-pressure grouting.

On the work in the northern portion of the City Tunnel, low-pressure grouting was carried on with three rigs, each consisting of the following equipment: One 30-h. p. electric motor; one Ingersoll-Rand 2-stage, belt-driven air compressor, capable of delivering 170 cu. ft. of free air per min. at 100 lb. pressure and 130 cu. ft. at 350 lb.; one geared pump, with a capacity of 50 gal. per min., driven by a 7½-h. p. electric motor; two grout tanks, two water tanks, a platform car for making connections to the grout pipes, and a platform mounted on two flat cars for the cement and sand. For high-pressure grouting the same rig was used, the compressor being adjusted to deliver air at 300 lb. per sq. in.

Grouting by Air and Water Pump.—In the Hudson Tunnel, where it was necessary to use grouting pressures of 700 lb. per sq. in., a combination of air and water-pressure pump was devised. A Canniff tank was partly filled with grout which was placed under a pressure of about 300 lb. by a Westinghouse booster connected as usual with the tank. Water was then pumped into the tank through another connection by a high-pressure Cameron pump, reducing the volume of air above the grout so as to raise the pressure to 700 lb. per sq. in., if desired. The discharge valve of the tank was then opened, and the pump was kept going in order to maintain the pressure while the grout flowed slowly into the space back of the lining. This arrangement worked very successfully. The advantage of a pump with a steady flow lies in the fact that there is no after-blast of air forced through the grout.

Paddle Mixing Machines.—In a few cases, paddle mixing machines which mix the grout mechanically instead of by air, were used. The results obtained by these machines are good when they are new, but the moving parts and the stuffing boxes soon give trouble on account of wear. The bad feature of both the air-stirring grout tanks and the paddle mixing machines is the blast of air which follows the discharge of grout. As already noted, it is very important to control the discharge of air in grouting in order to secure good results.

Grouting Pads.—The specifications called for the use of grouting pads for forcing grout into any porous concrete in the lining. These pads consisted of a steel casting measuring about 1 ft. by 2 ft. 3 in., containing a rubber gasket, and mounted at the end of a 5-in. extra heavy wrought-iron pipe. In using the grout pad, it was jacked against the porous spot in the lining by screw-jacks at the other end of the column, and the grout was then forced into the pad to impregnate the concrete. These pads were not successful, as they were clumsy to handle, the area covered by the pad was too small for effective work, and the gaskets would blow out. After a few attempts, their use was given up.

Only a few porous spots occurred in the concrete lining through which grout leaked during low-pressure grouting, and it was found that the most effective method of treatment was to wedge a board faced with oakum against the porous spot. The flow of grout soon stopped and the opening closed up.

Special Grouting in Pressure Tunnels.

Grouting Wet Stretch, Rondout Tunnel.—This pressure tunnel, 4.4 miles long, was constructed from eight shafts, varying in depth from 370 to 708 ft. The shafts and tunnel penetrated eight distinct formations, including shales, limestones, and quartz conglomerate. Several stretches of faulted zones and crushed rock were encountered in the excavation. Some strata were dry; others were very porous and water-bearing.

As already described, the tunnel north of Shaft 4 was driven on a 15% grade through a badly faulted and folded stretch in High Falls shale and Binnewater sandstone. The rock was porous and water-bearing, the inflow in a stretch of 350 ft. being approximately 2 000 gal. per min. at the time the tunnel was driven. The flow gradually

diminished until it was somewhat less than 1 000 gal. per min. at the time the lining was placed. The tunnel at this location is about 450 ft. below the surface of the ground. An attempt to shut off the inflowing water by grouting at the time of driving has been described.

It would have been difficult to take care of the water by means of drip-pans during the placing of the lining, as it poured into the tunnel over large areas. The plan adopted to protect the concrete during placing was to enlarge the excavation through the wet stretch in order to permit the erection of a light steel shell (see Fig. 5), inside of which a concrete lining, 24 in. thick, was placed. The shell was 175 ft. long, and consisted of circular ribs of 7-in., I-beams set 5 ft. center to center along the tunnel, with $\frac{3}{16}$-in. plates bolted to the outside of the ribs. The space between the shell and the rock, which averaged about 20 in., was packed solid with broken stone and subsequently grouted.

Most of the water entering the tunnel in this stretch was intercepted by 6-in. lateral pipes draining into an 8-in., spiral-riveted drain laid longitudinally along the bottom of the tunnel. This drain discharged through an 8-in. valve into a sump at the foot of the wet stretch. Water was also collected through holes drilled into the rock seams to depths of from 3 to 4 ft., into which 2-in. deep-seated pipes were set, which extended through the dry packing and the steel shell and drained into the tunnel. Pipes were also placed through holes in the shell to reach the dry-packed space for the purpose of grouting the packing behind the shell.

The concrete invert, which averaged about 3 ft. in thickness, was laid first on the floor of the tunnel over the 8-in. drain. The invert was placed in short stretches, 25 and 40 ft. in length, with tight wooden bulkheads and forms. Any leakage in the bottom was taken care of by pans and broken stone drains covered with cement bags. The placing of the invert was very difficult on account of the heavy flow of water from the bottom and sides of the tunnel. After the invert was placed, the steel shell was erected and the space between the shell and the rock was dry packed at the same time. The lining was then placed inside the shell without any trouble.

Preliminary to grouting this stretch, tight brick cut-off walls were built between the shell and the roof at each end of the shell. When grouting was started, the valve at the lower end of the 8-in. drain was

closed and the valves of all the pipes leading into the dry packing were also closed. The dry-packed space was quickly filled, forming a pond of still water, into which neat cement grout was injected. Experiments previously made showed that the materials in sand grout might separate if forced in under water. The deep-seated pipes and a few pipes at the top of the shell were left open to prevent accumulation of pressure.

Grouting of the dry packing was begun at the lower end of the shell. A battery of four Canniff grout tanks was used, the grouting of both sides of the tunnel being carried on simultaneously. The space behind the shell filled gradually, the level of the grout being indicated by opening pipes at various levels. A few of the deep-seated pipes flowed grout during this operation. They were closed, but were re-opened about 24 hours later and rodded out, after which they again flowed clear water, but in a somewhat diminished volume. In the whole section the inward leakage was reduced from 850 to 540 gal. per min. after grouting the dry packing, showing that the seams had been partly closed. The grout was injected into the dry-packed space, generally under low pressure, without causing undue agitation. The final grouting was done under a pressure of 90 lb. per sq. in. at the top of the shell. A total of 7 700 batches of neat grout (1.2 cu. ft. per batch) was forced into the dry packing, requiring ten shifts.

After the grout had set a few days, the deep-seated pipes through which several hundred gallons of water per minute issued into the tunnel, were grouted, beginning at the bottom of the shell and working up. During this process, varying consistencies of grout were used, the thinnest being $\frac{1}{4}$ bag of cement (about 24 lb.) to 30 gal. of water. After applying air pressure to clean out a rock seam, the thinnest grout was used first. If a considerable quantity, 20 to 30 batches, went into the rock under comparatively low pressure (100 lb., or less), the grout was thickened, 48 lb. to 30 gal. of water being used. If a considerable quantity of this grout was injected, the mix was again thickened, this process being continued until the tank pressure necessary for injection began to rise, when the mix was thinned slightly, and grouting continued until the pipe refused under 300 lb. pressure. The pipe was then drained out and plugged with thick neat cement grout under 300 lb. pressure. The maximum number of batches injected into a pipe was 533.

The grouting reduced the leakage on the incline from 850 to 3 gal. per min., and although the level of the water in the rock reservoir, as indicated by gauge readings, rose gradually after the completion of the grouting until the original ground-water level had been restored, there was no marked increase in the quantity of water entering the tunnel. Test holes drilled through the concrete lining and shell and into the rock showed that the rock surrounding the tunnel was impregnated with grout to a considerable depth. The success of the grouting at this location was due to the use of the deep-seated pipes, and, more particularly, to the method adopted whereby the dry-packed section outside of the lining and shell permitted the grout to come in direct contact with the surface of the porous rock.

Grouting Large Mud Seams, Rondout Tunnel.—In driving the tunnel through the Helderberg limestone south of Shaft 3, Rondout Tunnel, a crushed and faulted zone containing a number of large solution cavities filled with residual clay was encountered. The two widest seams were vertical and cut diagonally across the tunnel at an angle of about 45° to the tunnel axis. These two seams were 16 and 25 ft. wide on one side of the tunnel, but much narrower on the other side. They were filled with clay and disintegrated rock and, while excavating the tunnel, were lagged with steel channels supported by steel beams and timbering. The lining at the seams was strengthened by embedding steel shells, ½ in. thick and 41 and 59 ft. in length, in the concrete at each seam. In addition to reinforcing the lining, these shells also serve to prevent outward leakage from the tunnel.

In this stretch, a horizontal seam about 3 ft. thick also was encountered, the bottom of the seam lying about 4 ft. above the finished waterway. The mud layer was removed and the space above the waterway, about 7 ft. high, was subsequently filled with concrete. Above the horizontal mud seam was an opening in which were three rows of narrow vertical pockets, extending diagonally across the tunnel. These pockets were from 2 to 6 ft. wide and ran from a few feet to 30 ft. above the roof of the tunnel. Concrete was placed up to the top of the general roof excavation and the space above, including the narrow pockets, was filled with grout through pipes set in the concrete. A total of approximately 9 000 batches of 1:1 grout was placed in this stretch, requiring 13 days of three shifts each.

Rupture of Lining and Repair Work, Rondout Tunnel.—On the completion of the tunnel and shafts, they were filled with water for a hydrostatic test. Excessive leakage developed in the Rondout Tunnel and, after unwatering it, an examination showed that the lining was cracked in three stretches aggregating in length about 700 ft. The upper half of the tunnel section appeared to be lifted from the lower portion, water issuing in large quantities through the joint at the springing line and through other diagonal cracks in the lining.

To determine whether the lining had ruptured on account of unfilled places over the top of the arch, a test hole was cut, and several holes were drilled through the concrete to the rock in the roof. The space was found to be completely filled with grout, which was intact and in close contact with the rock. The rupture of the lining was due to the movement of the rock outside of the tunnel resulting from the application of hydrostatic pressure.

The tunnel was reinforced through the ruptured stretches by steel lining built up of rings of circular segments of 15-in., 55-lb. channels. A 2-in. annular space was left between the steel and the existing concrete lining. Preparatory to placing the steel, the cracks through which water issued were caulked with lead, thereby forcing the water into pipes drilled through the lining into the rock seams.

The space between the steel shell and the old concrete lining was first grouted. Every twelfth ring of steel had eight holes placed equally around the circumference, tapped and plugged for grouting. A 1:1 grout was used under a pressure of 30 lb. per sq. in. After this grout had set for one week, the deep-seated pipes leading to the wet seams were grouted. Very thin grout, ¼ bag to 35 gal. of water, was forced in until the hole refused at 300 lb. pressure. The thin grout was then drained out of the pipe, which was sealed with a thick batch. A total of 743 bags of cement was injected through 89 pipes. The leakage in this section before repairs were started amounted to about 400 gal. per min. After all the concrete was placed and all the grouting was finally completed, it was reduced to 2 gal. per min.

To protect the steel shell against rusting, and to provide a smooth waterway, a concrete lining, averaging 6 in. in thickness, was placed inside the steel shell. To fill the space above the arch, a grout and a vent pipe were placed in each pocket formed by the projecting flanges of the channels. A thick 1:1 mixture was used for the grouting, the

grout being placed by a Douglas hand pump. After this grout had set, and the surplus water was allowed to drain off, a second grouting was done with neat cement, completely filling the space between the lining and the steel shell.

Wallkill Pressure Tunnel.—This tunnel crosses the Wallkill Valley for a distance of 4½ miles, at an average depth below the ground surface of 350 to 450 ft. The rock penetrated was the Hudson River shale formation, and was generally sound and dry. For the greater length, the rock was a rather fine-grained shale with some beds of hard sandstone; for about a mile at the south end of the tunnel, it was mainly sandstone in very heavy beds. The dip of the rock for the greater length of the tunnel averaged about 45°, striking at an angle of about 30° with the tunnel line. At the north end, the bedding was vertical and at right angles with the tunnel axis; while for a considerable distance at the south end the beds were horizontal, causing high breakage above the roof.

The inward leakage when the tunnel was driven amounted to approximately 500 gal. per min. in the tunnel and 180 gal. per min. from the shafts. Although no great concentrated flows of water were met, certain stretches were water-bearing over large areas and the rock was extensively covered by pans to catch the water. Eight stretches of 1 000 ft. in length yielded an inward flow of more than 20 gal. per min. before grouting. The maximum inward leakage of ground-water was 90 gal. per min. for 1 000 ft. of tunnel.

In many places the water entered in fine sheets under head between the layers of shale. It was found to be practically impossible to force any quantity of grout into this rock on account of the fine openings between the thin layers. The wet areas were covered by drip-pans and grouted under high pressure with neat cement grout by filling the pans solidly full, sealing the fine openings, and effectively cutting off the water.

An interesting movement of the flow of water in the rock during grouting occurred between Shafts 1 and 2. The total leakage in this stretch just prior to starting grouting was 125 gal. per min. There was little diminution in the leakage after the low-pressure grouting and it was not until the high-pressure grouting plant had traveled a distance of about 3 000 ft. south from Shaft 1 to within about 2 000 ft. of Shaft 2, that any marked change in the inflow occurred. In

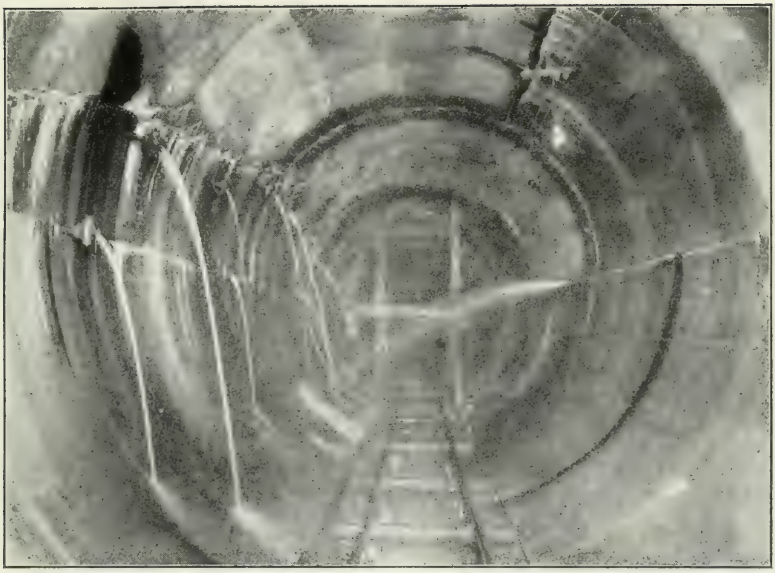


Fig. 7.—Broken Tunnel Lining, Rondout Pressure Tunnel. Cracks Due to Yielding of Rock. Leakage Reduced from 400 to 2 Gal. per Min. by Grouting.

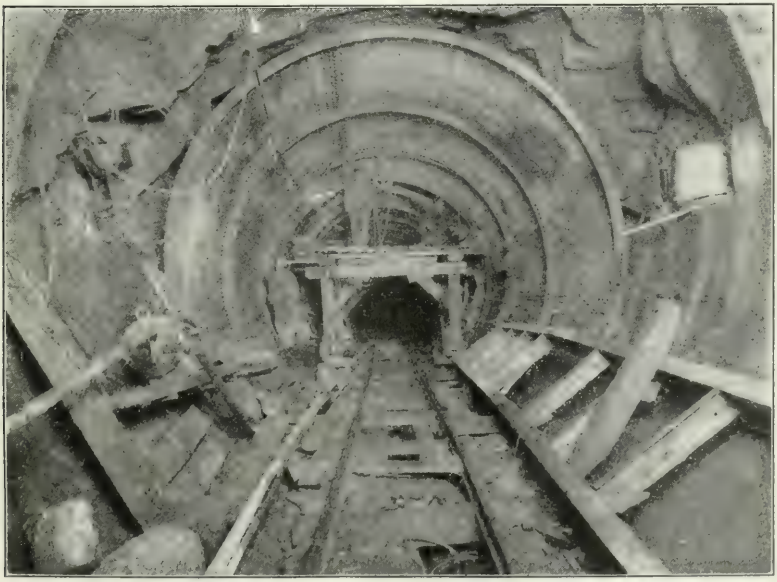


Fig. 8.—Steel Interlining Through Decomposed Rock, City Tunnel, Under 106th Street. Space Back of Lining Consolidated by Grouting.

two days of grouting, the leakage in this stretch dropped to 70 gal. per min.

Evidently, the water in a length of 3 000 ft. of tunnel, south of Shaft 1, was forced ahead during the high-pressure grouting, finding an outlet through grout pipes in the arch south of the grouting machine. There was a soft seam or plane of movement above the roof of the tunnel in this stretch, running nearly parallel with the tunnel, and apparently the water was forced through this seam during the grouting. On the completion of the grouting, the leakage in this stretch was reduced to 22 gal. per min.

Moodna Pressure Tunnel.—This tunnel is 4.8 miles long, and from 300 to 400 ft. below the surface of the ground, the maximum depth below the hydraulic gradient being about 600 ft. The tunnel penetrated Hudson River shale for about 60% of the distance and granitic gneiss for the remainder of the length. The shale was generally soft, although there were many layers of hard sandstone; the rock was much disturbed and without uniformity of attitude. The gneiss was hard and sound, but intersected in every direction by joint planes as a result of the disturbance due to a great overthrust fault which extends for many miles along the contact between the gneiss and the Hudson River shale formations.

The inward leakage for the entire tunnel, including the shafts, was 640 gal. per min. when the excavation was completed. The shale was wetter than the gneiss, but there were no large concentrated flows anywhere. Very few holes were drilled for deep-seated pipes, as practically no well defined water-bearing seams were encountered. In the shale, a drip-pan was placed about every 7 or 8 ft. of tunnel.

The rock throughout was generally firm and only 3% of the length of the tunnel required roof support. Although several zones of extensive rock movement were penetrated, there were no large openings or excessive inflows at these places, and special treatment was required in only one case, where badly crushed and unstable ground was encountered beneath the surface valley known as Pagenstecher Gorge.

In this stretch, a flow of about 100 gal. per min. was struck when the heading was first excavated. The water issued from a crevice 6 in. wide, marking a plane of movement and solution in the gneiss. The adjacent rock was crushed and disintegrated, and required a

steel roof support for a distance of 55 ft. along the tunnel. Following the first rush of water from the crevice, the leakage gradually diminished, as the underground storage was exhausted, until it was less than 10 gal. per min.

The rock broke very high and wide at this place, and, in making arrangements for grouting, long grout pipes were set to reach the top and sides of the cavity. The longest pipe extended 20 ft. above the roof of the tunnel. The dry packing between the steel support and the rock was first grouted, the operation being continued until the water-bearing crevice was filled. After the grout had settled and set, the long pipes were opened up and drained, and any remaining spaces were filled with grout under a pressure of 300 lb. per sq. in.

Repair Work in Moodna Pressure Tunnel.—The Moodna Tunnel connects at the south end with the tunnel under the Hudson River by a shaft on the west bank. Although the rock in Moodna Tunnel adjacent to the West Shaft was dense and solid when excavated, the application of hydrostatic pressure, when the tunnel was tested, caused a movement along the joint planes, with rupture of the concrete lining both in the West Shaft and in a stretch of Moodna Tunnel adjacent to the shaft. As the joint planes in the rock were closed and tight before the test pressure was applied, it would have been impossible to prevent the yielding of the rock by grouting, no matter how thorough and extensive, prior to the hydrostatic test. The cracks in the lining through which considerable water issued were subsequently caulked, and the rock was grouted. Later, it was decided, on account of the geological conditions which the test had revealed, to abandon 900 ft. of the tunnel adjacent to the West Shaft, and a new tunnel was driven 400 ft. below the old one. No further trouble was experienced at this location.

Hudson Pressure Tunnel.—This tunnel, placed 1 100 ft. below the river surface, penetrated sound Storm King granite throughout its entire length of 3 100 ft. Numerous black diabase dikes were found along the joint planes. The granite has shared with the entire Highland area in folding and faulting movements which broke up the rock into blocks separated by joint planes. In excavating the tunnel, some water from the river entered through this system of joints.

The grouting of a wet seam near the East Shaft in driving the tunnel has been described. Contrary to expectations, the leakage of

water into the tunnel was not excessive, and was taken care of in the usual manner by the use of drip-pans and deep-seated pipes. The total inward leakage after the completion of the concrete lining and prior to grouting was 160 gal. per min., the greater part occurring near the western portion of the tunnel.

As already described, some trouble was experienced in this tunnel on account of the cracking of the concrete lining in the side-walls, when the high grouting pressures, running up to 600 lb. per sq. in., were applied. After the cracks were cut out, the lining was replaced and again grouted under lower pressure, and no further trouble developed. As noted, a soft deposit was found between one of the drip-pans and the rock at one of the cracks which had been cut out, which probably prevented the filling of the space with grout. Several unusually large drip-pans were used in the wet stretches, and this may have been a factor in the cracking of the lining, as the very high grouting pressures were applied over large areas where the lining was not very thick.

No attempt was made to shut off all inflowing water in this tunnel by grouting, and the pipes which flowed water were fitted with bronze check-valves. These valves prevent the accumulation of rock-water pressure on the lining, when the tunnel is empty, by permitting the water to flow freely into the tunnel, and also, by closing tightly, prevent the loss of water from the tunnel when in service.

The high pressures necessary for grouting this tunnel were obtained by the use of a high-pressure pump which discharged the grout in the grout tank by pumping water into the tank. This tunnel proved to be remarkably tight when tested, and the loss of water when in service is very little.

Repairs in Breakneck Pressure Tunnel.—This tunnel, 780 ft. long, extends from the East Shaft of the Hudson Tunnel to an uptake shaft on the north slope of Breakneck Mountain. It penetrated the same granite formation as the Hudson Tunnel, and the rock was sound and dry. In the portion adjacent to the East Shaft, where the rock cover is comparatively shallow, cracks developed in the tunnel lining, as well as in the upper portion of the East Shaft, when a hydrostatic test was made, due to yielding of the rock along the joint planes. The lining was repaired by drilling holes about 10 ft. deep through the concrete and into the rock, and injecting thin

grout under pressure. Most of the holes took little grout, but there were a few into which a considerable quantity was forced.

The grout was mixed in the proportions of 25 lb. of cement to 20 gal. of water, and, in some cases, even thinner. Water was used for washing out the seams and frequently permitted further grouting after a hole appeared to be plugged. One hole, in the vicinity of a fractured zone in the rock, took 5 217 batches of grout containing 328 bbl. of cement. This hole was almost plugged several times, but was opened by flushing with water.

During grouting, the grout did not appear in any of the other holes in the vicinity of the pipe grouted, but was evidently forced some distance from the tunnel. Only four holes took more than 25 bbl. of cement. A total of 556 bbl. of cement was used in grouting the tunnel and 39 bbl. in the East Shaft. The repairs were successful in reducing the outward leakage to a small quantity.

City Pressure Tunnel.—The City Tunnel forms the great distribution main within the city limits. It is 18 miles long, and is located beneath the streets and parks of the Boroughs of The Bronx and Manhattan, extending from the Hill View Reservoir, just north of the city line, to two terminal shafts in Brooklyn. The tunnel was constructed from 24 shafts, varying in depth from 218 to 757 ft.

The tunnel was placed everywhere at a depth of not less than 150 ft. below the surface of sound rock. The rock is at or near the surface over the greater part of the line, and more than half of the length of the tunnel is little more than 200 ft. below the surface of the ground. In order to secure the required depth of sound rock cover over the tunnel under the Harlem and East Rivers, the tunnel was placed 330 and 700 ft., respectively, below the surface.

The formations penetrated in the City Tunnel, namely, Yonkers and Fordham gneiss, Inwood limestone, Manhattan schist and Ravenswood diorite, are generally sound and durable rocks, but were found in many places to be disturbed by folding and faulting, with extensive crushed zones. Intrusions of pegmatite were frequently met. The underground waters circulating through channels formed by mechanical fracture and solution have caused chemical changes in many stretches, with resulting alteration and weakening of the mineral constituents of the rock. For the greater part of its length, the

tunnel was driven approximately parallel with the strike of the rock, the dip being generally nearly vertical.

With the exception of two or three stretches, where heavy flows of water were encountered, the rock throughout the length of the tunnel was generally dry, and seeping water was taken care of by the methods used in the other tunnels. On account of the location of the City Tunnel, in the heart of the city, unusual precautions were taken in handling the water and grouting, in order to reduce, as far as practicable, the loss of water out of the tunnel, as it was evident that excessive leakage would be exceedingly troublesome and might result in the settlement of buildings.

Grouting under Harlem River, City Tunnel.—The tunnel under the Harlem River Valley penetrated Inwood limestone, a white crystalline marble, solid and dry for the greater part, but, for a distance of about 200 ft., a blocky stretch with water-bearing seams was encountered, yielding a flow of about 200 gal. per min. A number of large crevices, several inches wide, extended above the roof and from the sides of the tunnel. Grout pipes, some 20 ft. long, were inserted in these crevices.

All the leaks in the sides and roof of the tunnel were panned, while, in the bottom, where considerable water issued, a broken stone drain covered by pans was laid below the invert concrete. The inflowing water was drained by a longitudinal pipe on the bottom. During the placing of the invert concrete, the water was kept below the top of the stone drain by pumping through the longitudinal pipe and pumping was continued until the concrete had set.

The grouting in those portions of the tunnel adjacent to the wet stretch was completed before any leakage in that stretch was cut off. Grout was first forced into the bottom drains and pans, the upper pipes being left open to relieve the water pressure. Connections were made successively to the pipes at higher levels, until the crevices in the roof were filled. The final grouting was done under a pressure of 300 lb. per sq. in., using thick cement grout. The grouting was very successful, the inward leakage in this stretch being reduced to 3 gal. per min.

Grouting the Wet Stretch near Shaft 23, City Tunnel.—The wettest stretch in the City Tunnel was encountered near Shaft 23, where, in a crushed zone in granodiorite, a flow of 600 gal. per min., issuing from one seam, was struck when the excavation was first made. This flow

had diminished to 325 gal. per min. at the time the concrete lining was placed. On account of the large flow, it was decided to use a light steel shell to protect the concrete, instead of the customary drip-pans.

The construction was similar to that in the Rondout Tunnel and consisted of ribs built up of 4 by 3 by ½-in. angles spaced 5 ft. center to center, on which plates ⅛ in. thick were placed. Each rib was made in four segments, the bottom being embedded in the invert concrete. Where the rock showed any leakage, the space between the steel shell and the rock was filled with dry packing, elsewhere the steel plates were omitted, and the concrete lining was placed directly against the rock. Dry packing was placed over about 60% of the area in the 90 ft. of tunnel in which this special construction was used.

The water in the wet stretch was drained by a 10-in. tile pipe, laid with open joints on the floor of the tunnel and surrounded by broken stone. Above the top of the broken stone, corrugated sheet iron was placed, on which the invert concrete was laid. The 10-in. tile drain had a slope of 6 in. in 100 ft. and discharged the water into a sump at the end of the wet stretch. From the sump, the water was pumped into an 8-in. discharge line in the tunnel by a motor-driven centrifugal pump.

After all the other grouting in the tunnel between Shafts 23 and 24 had been completed, that in the wet stretch was begun by first connecting to the pipes in the bottom drain and then successively to the pipes at the higher levels, raising the grout up to a maximum of 150 lb. per sq. in. By this means, most of the grout was placed against a back pressure of 50 lb. per sq. in. or less, the gauge registering finally 90 lb. per sq. in. when the grouting plugged it.

In order to prevent the accumulation of ground-water pressure, all the pipes except those in the invert were left open, or were opened up after the grout had set. After the deep-seated pipes were grouted under a pressure of 300 lb. per sq. in., all the pipes were opened and connected to for high-pressure grouting. A total of 217 batches of neat cement grout, consisting of 1 bag of cement (95 lb.) and 6 gal. of water, was used for the high-pressure grouting in this stretch. Very successful results were obtained in cutting down the inward leakage which was reduced by grouting to 6 gal. per min. for the entire stretch of 3 600 ft. of tunnel between Shafts 23 and 24. Inspection of the tunnel after a

hydrostatic test under the operating head, showed no cracking or inward leakage in this stretch.

Repairs in City Tunnel Between Shafts 18 and 19.—The portion of the City Tunnel underneath Broadway, in the vicinity of 23d Street, was excavated through fairly sound Manhattan schist, with no inward leakage. When the tunnel was subjected to a hydrostatic test, some water appeared at the surface in Madison Square Park and in some of the cellars near-by. Although the outward leakage was not excessive, it was decided to make an effort to cut down the quantity as much as practicable. On unwatering the tunnel, it was found that, due to movement of the surrounding rock, the construction joints of the tunnel lining had opened up and, in addition, some of the lining was cracked. Four stretches of tunnel aggregating 1 350 ft. in length were affected.

The method of repair adopted was to consolidate the rock surrounding the tunnel by grouting and, in addition, to line with sheet copper two stretches aggregating about 1 200 ft. of tunnel. A large number of holes, 2 in. in diameter, were drilled through the concrete lining and into the rock, the size of the tunnel limiting the maximum depth of the holes to 10 ft. In the 1 350 ft. of tunnel grouted, approximately 10 000 lin. ft. of holes were drilled. Pieces of 1½-in. pipe, 15 in. long, were fitted into these holes.

Before grouting, grooves were cut in the joints and cracks through which water entered the tunnel and these joints were caulked with lead wire. The customary grouting equipment was used for this work, air at 90 lb. being furnished by a compressor at the top of the shaft. Higher pressures were obtained by Westinghouse "boosters".

After connecting to a hole, grouting was continued, until the hole refused grout under 300 lb. pressure. Any leaks that developed during the grouting were located, and, after the grout had set, additional holes were drilled at the place of leakage. The process of drilling and grouting a section was continued until test holes, drilled 10 ft. deep, did not leak or take grout. The stretches in which the lining was cracked were grouted three times, a portion of one being grouted five times.

The mix ordinarily used consisted of batches containing 25 lb. of cement to 20 gal. of water. If a hole into which 75 batches of this consistency had been injected, continued to take grout freely, the mix was thickened to 25 lb. of cement and 5 gal. of water; but if the hole showed

a tendency to plug, the thin mix was again used. A total of 6 000 bags of cement were used in grouting the 1 350 ft. of tunnel.

When the grouting was started, the inward leakage amounted to 30 gal. per min., practically all of which was cut off by grouting. The ground-water head against the lining during grouting, as indicated by gauges, was 78 lb. per sq. in. After the tunnel was again placed in service, there was practically no leakage at the surface of the ground.

Grouting in Shafts.

In connection with the construction of the pressure tunnels of the Catskill Aqueduct, fifty-five shafts were sunk. Some of these are permanent, forming part of the waterway; the others were sunk only to expedite the tunnel excavation, and subsequently were closed by concrete plugs. The shafts varied in depth from 106 ft. to a maximum of 1 187 ft., the average being about 400 ft.

As in the case of the tunnels, the shafts penetrated a wide assortment of rock. With a few exceptions, however, no great difficulties were experienced in sinking them, as the rock encountered was generally sound and free from large quantities of water. In some of the shafts where the inflow of water was excessive, the rock seams were grouted during sinking in order to cut off or reduce the flow. In general, the water-bearing ground did not extend to any great depth, but was found in zones about 20 to 30 ft. thick, so that, after the wet portion of the shaft was grouted and passed, no further grouting was necessary in the remainder of the shaft.

Shaft sinking is slow and expensive when large quantities of water have to be handled. The water collects in the bottom, making it troublesome to drill and to muck the broken rock, thereby delaying the work. The maintenance and operation of pumps also interferes with the excavation and increases the cost. The method of grouting water-bearing seams in a shaft during excavation, to cut off the inflowing water, has been used to some extent, although the process has been more fully developed and applied during the last few years.

The method usually adopted in grouting a wet zone is to drill a number of vertical holes in the bottom of the shaft, about 10 to 20 ft. in depth and located generally from 5 to 10 ft. around the periphery of the shaft. Short lengths of pipe fitted with valves are set into the drill holes, and the drilling is done through the valves which can be quickly closed if a heavy flow of water is struck.

After the holes have been drilled, grout is forced into the rock seams through them, by the usual grouting equipment. As in other grouting, a thick grout is best for filling large cavities, while a thin grout, as thin as 5 lb. of cement to 25 gal. of water, is necessary during the early stages of the injection of the grout, in order to make the cement carry as far as possible into the fine seams and avoid the plugging of the seams close to the drill hole. The matter of adjusting the consistency of the grout is important to insure success. After the grout has set up, and the water is cut off, sinking is resumed, using light shots to avoid opening up new fissures in the rock. The method of grouting outlined was used in sinking through wet ground in several of the Catskill shafts. The sinking in these cases would have been much more difficult, slower, and more expensive, had pumping alone been depended on.

Shaft 4, Rondout Pressure Tunnel.—This shaft, 500 ft. deep, was one of the construction shafts for the Rondout Pressure Tunnel. It was rectangular in shape and was timbered as it was sunk. This was the wettest shaft on the Catskill work, and the grouting of the water-bearing strata encountered in its excavation was very successful in cutting down the inflow of water, although pumping had to be carried on continuously during the sinking as additional wet seams were struck. A detailed description of the sinking of this shaft has been given in a paper* by John P. Hogan, M. Am. Soc. C. E.

Preliminary exploration by borings showed that the shaft would penetrate Binnewater sandstone and High Falls shale at a depth of about 200 ft. The rock cores from these beds showed a very porous condition, and pumping experiments indicated that the rocks were freely water-bearing. It is probable that considerable time would have been saved and expensive pumping eliminated during the sinking of the shaft if 4-in. holes had been drilled close together around the perimeter of the shaft when the water-bearing strata were reached, and the rock thoroughly solidified by injecting grout into the seams through these holes. The shaft could then have been excavated in the dry.

However, the drilling of the large holes necessary to insure the complete success of the grouting under such a method would have been a slow and expensive procedure, and experience at the time did not

* "Sinking a Wet Shaft", *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 398.

recommend it as the best method of meeting the situation. The handling of water by pumping was familiar to the shaft sinkers who kept on drilling and mucking until stopped by an excess of water, even though the shaft was flooded several times before the grouting was done.

The shaft was dry until a depth of 215 ft. was reached, when the water-bearing Binnewater sandstone was encountered. At a depth of 260 ft., the flow had increased to 225 gal. per min. While drilling the next round of holes, one of the holes struck such a strong flow that the men had to leave hurriedly, and the shaft was flooded to within 70 ft. of the top. The maximum flow of water entering through the drill holes in the bottom was estimated to be approximately 600 gal. per min.

After a long delay, the shaft was pumped out, and the hole in the bottom was plugged with a 2-in. nipple and gate-valve. Drilling was then resumed in the bottom, through valves on pipes set into the drill holes, the valves being shut as water was encountered. On account of trouble with the pumps and the suction and discharge pipes, the shaft was again flooded twice in succession and each time recovered.

The last holes drilled in the bottom indicated that for the next 8 ft. the ground to be excavated was very porous, with open seams as wide as 8 in. It was feared that, when the next shot was made, there would be such a great inflow of water as to flood the shaft again. It was then decided to attempt to cut off the water by grouting the water-bearing seams.

For this purpose, twenty-seven vertical holes were drilled from 14 to 20 ft. deep, around the bottom of the shaft, each hole being fitted with a pipe and valve. A total of about 3 000 bags of cement was forced into these holes. At first, the grout leaked back into the shaft, but after some experimenting, the seams were finally clogged by mixing bran, oats, and ground horse manure with the cement. This grouting was so successful that it was decided to attempt to grout the remainder of the water-bearing strata in a single operation down to the top of the Shawangunk grit, a hard quartz conglomerate encountered in sinking the lower portion of the shaft. Six diamond drill holes, 1 in. and 2 in. in diameter, were drilled around the perimeter of the shaft for a depth of 100 ft. The holes were grouted under a pressure of 275 lb. per sq. in., but only 175 bags of cement could be forced in. After further delays, due to pump troubles, sinking was resumed. In the first 15 ft. exca-

vated, numerous seams well filled with grout were encountered; the maximum thickness of the grout was 8 in., in an irregular open bed of sandstone. The leakage into the shaft at this time was 225 gal. per min., which increased to 350 gal. per min. at a depth of 280 ft. The water continued to increase in quantity to 525 gal. per min. at a depth of 320 ft., and was strongly charged with H_2S gas, which attacked the men's eyes and caused abrasions to develop into sores.

In the meantime, the pumps continued to give trouble and with the shaft filled with no less than ten of them at one time, and with air and discharge lines, it was almost impossible to carry on the sinking operations. Due to the breaking down of one of the station pumps in the shaft, the latter was flooded for the sixth and last time. After it was again unwatered, a large pump chamber was excavated in the side at a depth of 310 ft., and three steam pumps having a combined capacity of 1 050 gal. per min. were installed therein.

Sinking was then resumed (a pilot drill hole being kept well below the bottom) and all seams were grouted, none of them taking more than 50 bags of cement, until the grit was reached. At the contact with the grit, an additional flow of 125 gal. per min. entered through an old drill hole, increasing the total inflow to 850 gal. per min. The flow was cut off by injecting 350 bags of cement under a pressure of 275 lb. per sq. in. All water-bearing seams in the grit were grouted, the maximum quantity of cement used at any point being 100 bags. When the bottom was reached, the total inflow into the shaft was 610 gal. per min.

The method of grouting used in this shaft was, in general, as follows: A 2-in. pipe provided with a long nipple and a gate-valve was driven into the drill hole as far as possible. To obtain a tight packing around the pipe, wooden wedges, oakum and cement were freely used, and horse manure mixed with the cement was also frequently used to clog the openings. A battery of four to six Canniff grout tanks was set up at the top of the shaft and connected with the hole by a 2½-in. pipe. The air pressure for grouting varied from 100 to 275 lb., the former being obtained from the main air line, and the latter from a small Westinghouse "booster." A thin mixture was used at first, and the quantity of cement was increased gradually; if this mix continued to flow freely, sand was added until the pipe began to clog.

The grouting in this shaft was very successful, at one time cutting off a leakage of more than 400 gal. per min. A total of 1 163 bbl. of cement was used in the grouting in the shaft. The holes drilled for grouting were generally 30 ft. deep, which depth was probably too great, as the grout would fill the wide horizontal seams, while the finer seams, cut nearer the bottom of the drill holes, would only be grouted for perhaps a few inches from the holes, due to the fact that in these seams the grout would set hard, while the wide seams were taking grout freely.

It is believed that the grouting would have been even more effective if shallower holes, about 12 ft. deep, had been drilled, as such holes would have cut a smaller number of seams, with the result that the rock would have been more thoroughly impregnated, and the grout would have traveled farther out from the shaft. On other work the best results have been obtained by alternately drilling and grouting, using shallow holes for the purpose.

City Tunnel Shafts.—All the shafts of the City Tunnel were lined with concrete. Although considerable leakage was encountered in several during the sinking, the inflow was either completely cut off or considerably reduced by grouting the water-bearing seams. If these wet shafts had been timbered instead of having been lined with concrete, the excavation would have been more difficult on account of the excessive quantity of water that would have had to be handled.

Shaft 4, City Tunnel.—This shaft, on the south side of Jerome Park Reservoir, was sunk to a depth of 242 ft. in Fordham gneiss. The rock was sound, and the shaft was dry until a depth of 150 ft. was reached, when a flow of 150 gal. per min. was encountered while drilling holes in the bottom. In a short time, the quantity diminished to a constant flow of about 50 gal. per min. After the shaft, which had been flooded, was pumped out, the drill holes were plugged and the concrete shaft lining was placed to within a few feet of the bottom.

The drill holes were then grouted under an air pressure of from 100 to 225 lb. per sq. in., using a mix of about 16 lb. of cement to 1 gal. of water. A total of 46 bags of cement was used in this grouting, and the leakage was entirely cut off. Water under a pressure of 70 lb. per sq. in. was also struck at a depth of 181 ft., and the flow at this level too was cut off by grouting.

Pilot holes, 14 ft. deep, were then drilled and all struck water, which in some cases washed 1-in. fragments of rock from the drill holes, indicating the presence of a disintegrated seam. These holes were grouted, but they took very little cement. The shaft was sunk another 5 ft. without increase of water. The bottom of the shaft was then in disintegrated rock well grouted.

The next step was to drill a large number of vertical holes, 18 in. apart and 16 ft. deep, around the periphery of the shaft. The holes were grouted, one at a time, each hole taking about two batches of grout, except some at one side of the shaft through which the leakage was about 100 gal. per min. The water from these holes could not be shut off, and the leakage was taken care of by pumping. As the shaft was deepened, no increase of flow was encountered, and sound rock was reached at a depth of 200 ft.

The water-bearing ground below the 181-ft. depth showed a crushed fault zone with a maximum thickness of 8 ft. on one side, below which the rock was disintegrated to a depth of 4 ft. A total of approximately 1 000 bags of cement was used in grouting this stretch, and although the disintegrated ground was found to be well impregnated with grout, the flow of water could not be entirely cut off. Thick grout was generally used, but it is believed better results might have been secured by the use of thin grout. After the shaft had been completely lined, the water-bearing seams were grouted through pipes set in the lining, and the leakage was reduced to 3 gal. per min.

Shafts 20 and 21, City Tunnel.—Shaft 20, 750 ft. deep, in the lower part of Manhattan Island, was excavated in Fordham gneiss which was generally dry. No trouble with water was experienced until a depth of 425 ft. was reached, when a flow of 30 gal. per min. was struck in the drill holes in the bottom of the shaft. Deep holes were drilled, pipes inserted, and a concrete blanket 2 ft. thick was placed over the bottom of the shaft. Grouting was started too soon, before the concrete had set hard, and it was necessary to add another 2-ft. layer of concrete. After a lapse of 48 hours, the holes were grouted under a pressure of 300 lb. per sq. in., and the water was cut off. No further trouble was had in excavating the remaining depth of the shaft.

Shaft 21 was sunk in granodiorite to a depth of 757 ft. at the East River, in the lower part of Manhattan Island. The rock was hard and

seamy, and wet zones were encountered about every 60 or 80 ft. The wet seams were grouted successfully in every case. A concrete blanket was placed over the seam to be grouted, as at Shaft 20, to retain the grouting pressures, and the results were satisfactory. The concrete lining in the shaft was placed in stretches of about 50 ft., as the sinking progressed, and any water entering through the grout pipes set in the lining was grouted off, in order to keep the bottom of the shaft as dry as possible.

Shafts 22 and 24, City Tunnel.—Shaft 22, in the Borough of Brooklyn, was sunk through gneiss and granodiorite. Only small quantities of water were handled, except at Elevation —430, where a flow estimated to have been 200 gal. per min., was encountered in a drill hole. The flow was controlled by inserting a pipe provided with a valve in the hole, after which other holes from 5 to 20 ft. deep were drilled to locate the extent of the water-bearing area, which was found to be on one side of the shaft.

The first attempt at grouting was unsuccessful, due to the large number of seams. A concrete blanket was then placed around two of the pipes, in the wet portion of the shaft, and the water was cut off by grouting under a pressure of 350 lb. per sq. in., only 6 days after it was first struck. Subsequent excavation showed that seams not more than $\frac{1}{16}$ in. wide were filled completely with cement. These seams were sharply defined cracks in the hard rock and were cut by the drill holes in such a manner as to provide good circulation of the grout, with the result that the grouting was successfully completed in a few shifts.

The granodiorite and gneiss through which Shaft 24 was sunk was blocky and more or less crushed. Little trouble was encountered in handling the water in the shaft, except in a stretch of 40 ft. at a depth of about 300 ft., where a flow of 240 gal. per min. was struck by one of the drill holes. The shaft was flooded, and it became necessary to install a large pumping plant to unwater it. After it was pumped out, the hole through which the water issued was plugged, and fourteen holes were drilled around the periphery of the shaft, varying in depth from $7\frac{1}{2}$ to $17\frac{1}{2}$ ft. Through pipes fitted into these holes, 55 cu. yd. of grout was forced into them under pressures as high as 325 lb. per sq. in.

The grouting of the wet seam caused the water to rise in the rock and flow from the smaller seams above. Holes were drilled into these seams, and grout under low pressure was forced into them, wedges of soft wood being driven into the seams where necessary to prevent the grout from escaping into the shaft. The leakage from these seams was reduced from 50 to 15 gal. per min.

On resuming excavation, it was found that the seam which yielded the large flow had been successfully grouted and gave no further trouble, but, after the smaller seams above had been closed, the rock in the shaft scaled so badly that it was necessary to suspend sinking and place the concrete lining. The leakage remaining was taken care of by pipes set in the concrete and subsequently grouted.

Grouting Behind Caissons, City Tunnel.—Before ledge rock was reached, the six southerly shafts of the City Tunnel had to penetrate through a deposit of water-bearing sand, varying in depth from 43 to 150 ft. The earth portions of these shafts were excavated by reinforced concrete caissons sunk by compressed air.

In order to effect a tight seal at the surface of the rock, the caissons were sunk from 2 to 5 ft. below the lowest point of sound rock around the cutting edge, and the space between the caisson and the rock was grouted through pipes set in a concrete bench on which the cutting edge of the caisson was sealed. Pipes were also placed through the concrete above the bottom of the caisson for the purpose of grouting the sand surrounding the caisson. The work was successful, as the leakage from the seal, after taking off the air, was only 7 gal. per min. at the deepest caisson, that at Shaft 23, where the hydrostatic head was more than 100 ft., and, subsequently, all this leakage was grouted off.

The experience in grouting the sand behind the caissons indicated, as found on other work, that little can be done to consolidate a mass of sand by attempting to fill the voids in it by grouting, but very good results can be obtained by grouting in filling voids due to loss of ground.

Grouting Closures of Construction Shafts.—The construction shafts which were sunk solely to expedite the excavation of the pressure tunnels, were closed by filling them, for a depth of about 50 ft. above the waterway of the tunnel, with concrete. These shafts were rectangular in shape, measuring about 10 by 22 ft., and were tim-

bered. Before the concrete plug was placed, the timbering in the bottom 50 ft. was removed, and the concrete was placed solidly against the rock.

It was realized that the concrete in the plug might shrink and leave a space between it and the rock through which the water from the tunnel would escape to the surface. In order to permit the filling of the shrinkage space, grouting channels were left entirely around the shaft at the top of each lift or at vertical intervals of about 10 to 15 ft. These channels were formed by placing sheets of tin or sheet iron around the shaft, the bottom edges of the tin being left in the fresh concrete, while the upper edges were wedged tightly against the rock.

Each grouting channel was provided usually with three 2-in. pipes set in the concrete; one, acting as a vent or drain pipe, extended down to the tunnel and two were carried up to the top of the plug. The channel was kept from clogging during the placing of the concrete by flushing it with water through the pipes leading to the top of the plug. After the concrete had set and shrinkage had taken place, the channels were grouted from the top of the plug, beginning with the lower one and working up.

The usual procedure before injecting the grout was to apply air pressure to each of the two pipes leading up from the grouting channel. While one pipe was being blown out, the other was kept closed, so as to wash any water or accumulation through the down drain pipe into the tunnel. Neat cement grout was then forced into one of the pipes until it showed at the vent pipe in the tunnel; the vent was then closed, and grouting was continued until the grout flowed from the top of the other up pipe. The latter was then closed and grouting continued, until no more grout could be forced in under a pressure of 300 lb. per sq. in. The high pressure was then applied to the other up pipe, and the process was continued until all the channels were grouted.

There is some question as to whether grout can be forced into the fine shrinkage space that may develop between the concrete plug and the rock. In Shaft 3, Rondout Pressure Tunnel, where it was necessary to excavate the plug in connection with the repair work in the tunnel, there were no indications of any layer of grout between the concrete and the rock. The concrete was found to adhere closely

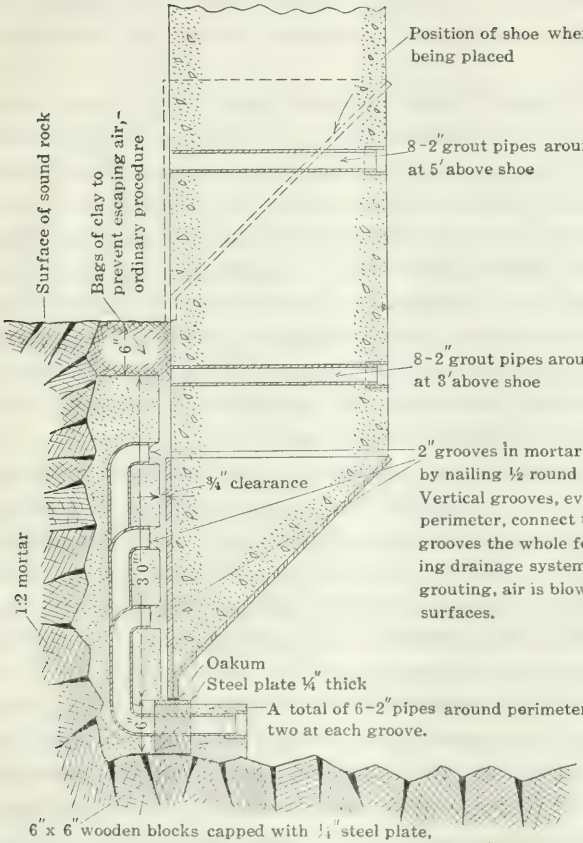

Position of shoe when mortar is being placed
8-2" grout pipes around perimeter at 5' above shoe
8-2" grout pipes around perimeter at 3' above shoe
2" grooves in mortar collar formed by nailing ½ round strips to form. Vertical grooves, every 6 ft. around perimeter, connect the horizontal grooves the whole forming a grouting drainage system. Just prior to grouting, air is blown in to clean surfaces.
¾" clearance
Surface of sound rock
Bags of clay to prevent escaping air,- ordinary procedure
6"
1:2 mortar
3'0"
Oakum
Steel plate ¼" thick
A total of 6-2" pipes around perimeter, two at each groove.
6"
6" x 6" wooden blocks capped with ¼" steel plate, placed at 4' intervals to receive shock of dropping caisson. Five inches of oakum placed under the entire perimeter prior to dropping, to insure water tightness. After caisson is dropped, grouting is immediately started.
3'0" collar may be reduced to 2'0" collar depending on rock conditions and ground water level.
CATSKILL AQUEDUCT
CITY TUNNEL
CONTRACT 67 SECTION 8
COMPRESSED AIR WORK
SEALING CAISSON IN EARTH TO ROCK
SCALE ⅜" = 1'

FIG. 9.

to the rock surface. Grouting pans are of advantage, however, in protecting the concrete from water seeping into the shaft. If the water is not collected, it will find its way through the concrete, or between the concrete and the rock, resulting in injury to the concrete or to the bond.

In Shaft 6, Rondout Pressure Tunnel, 63, 45, and 53 batches of neat cement grout were forced into the lower, middle, and top grouting channels, respectively, of the shaft plug. The total volume of grout injected, namely, 161 batches, corresponding to about 200 cu. ft. of grout, exceeded the estimated volume of space in the grouting channels, so that it is believed that some of it penetrated into the rock seams. There was also indication of this during the high-pressure grouting of the top channel. Grout appeared in the rock seams above the plug and also at the top of the plug at the contact between the concrete and the rock, 19 ft. above the top channel. The grout leaks clogged up as the air pressure was continued.

In grouting many of the construction shaft plugs, no trouble was experienced, after connecting to a pipe leading to a grouting channel, and closing all other vents, to force air either through the rock itself or through the contact between the concrete and the rock. The fact that the air pressure could be kept at 100 lb. during the previously mentioned test, or raised to a higher pressure, showed that the seams and openings were very small.

It is important while removing the shaft timbering, in connection with the placing of the concrete plug, to take down all loose rock and place the concrete against the solid rock. It is possible that the passage of air occurring during the tests previously noted, took place through loose rock seams instead of between the concrete and the rock. Not only should loose rock be removed, but also all rock that sounds "drummy" when pounded.

The concrete plugs at the bottom of the construction shafts proved to be very tight against outward leakage.

Grouting Closures in Riser Valve Shafts.—The City Tunnel shafts through which water is supplied from the tunnel to the valve chambers at the top of the shafts, are provided with riser valves which are about 100 ft. below the top of sound rock. These valves are designed to close automatically if a break occurs in the valve chamber or in the street main. Below them, the waterway is a 14-ft., or larger,

concrete-lined shaft, and, above them, the 4 or 6-ft. riser is a mortar-lined steel pipe, embedded in a solid plug of concrete forming the shaft closure. The concrete plug was placed inside of an outer shaft lining of concrete, the latter having been placed during the sinking of the shaft.

Grouting rings or channels were formed at frequent intervals, when placing the plug concrete, to permit the grouting of the shrinkage space between the new and the old concrete. These channels were grouted after the concrete had been in place a sufficient length of time to insure complete shrinkage.

The procedure in Shafts 15 and 16 was typical of the practice in the City Tunnel shafts. In these two shafts, three grouting channels were formed between the concrete plug and the outer lining, the lowest one being about 5 ft. above the point where the plug started, and the others about 10 and 20 ft., respectively, above the bottom one. The channels were formed at the end of the day's work by placing segmental timbers of 4 by 4-in. cross-section in the top of the green concrete, around the perimeter of the plug.

Two 2-in. pipes, placed diametrically opposite, were carried from the bottom of the plug to the bottom of each grouting channel. Before starting to place the next lift of concrete, the channel was covered with sheet iron bent against the outer lining and covered with a sloping layer of stiff concrete about 1 ft. thick. The latter protected the sheet iron and kept it from being displaced.

After the concrete plug had set six weeks, the channels were grouted. The general procedure in each case was to blow out the pipes and channels with compressed air at pressures of 50 to 60 lb., after which water was forced through, and, finally, grout was injected under pressures up to a maximum of 150 lb. per sq. in. The grout used varied from a mix of 1 volume of cement to 8 volumes of water to one of equal volumes of cement and water. The experience at these shafts showed a slight passage of water and grout from channel to channel, but no evidence of leakage showed around the plug at the top of the shaft.

In Shaft 24 of the City Tunnel, four grouting channels were provided in connection with the placing of the concrete plug which is 197 ft. deep and 24 ft. in diameter. To each of the two lower channels, three 2-in. pipes were carried from the shaft below the plug,

and from the two upper channels, three pipes were carried to the top of the plug. Compressed air was forced into the pipes of the lower channels before any grouting was begun. The air circulated freely between the plug and outer lining, appearing at the surface.

The bottom channel was grouted first, the grout being forced into the pipes until they refused it under 300 lb. pressure. The grout from the lower channel flowed upward into the next channel and its pipes, but reached the top of the plug through the pipes of the upper channels. Connection was made to each pipe, the lower channels being completely grouted before the upper ones were begun.

The results obtained at all the City Tunnel shafts have been very satisfactory, as there has been no leakage into any of the chambers at the top of the shafts. At some of the shafts, the pressure of the water at the level of the chamber floor is about 110 lb. per sq. in.

Grouting Under the Foundations of Dams and Dikes.

Grouting Seams in Rock Under Olive Bridge Dam.—The masonry portion of this dam is about 1 600 ft. long, and rises to a maximum height of 210 ft. above the bed of Esopus Creek. The bluestone layers underneath the creek were excavated to sound rock at a maximum depth of about 25 ft., and, in addition, a cut-off trench about 20 ft. wide was excavated, to a maximum depth of 40 ft., across the creek channel just below the up-stream edge of the dam. Below the top layer of bluestone, alternating layers of slate, shale, and bluestone were found, the beds varying in thickness from 1 to 15 ft.

Before excavation for the dam was started, the rock underlying the foundations was explored by means of fifteen diamond drill borings from 40 to 100 ft. deep, put down in the bed of the creek and the adjacent rock sides. The holes were tested by hydrostatic pressure tests for the purpose of locating the position and permeability of the seams in the rock. These investigations showed two definite planes of leakage at depths of 40 and 60 ft. below the bed of the creek.

To prevent the flow of water through the open seams, a total of 217 holes, 2½ in. in diameter, were drilled to about 1 ft. below the seams for the purpose of grouting them. The lower seam was reached by a row of 45 holes, spaced about 5 ft. apart, drilled from the bottom of the cut-off trench. For grouting the upper seam, seven rows containing 172 holes, also spaced 5 ft. apart, were drilled from the bottom

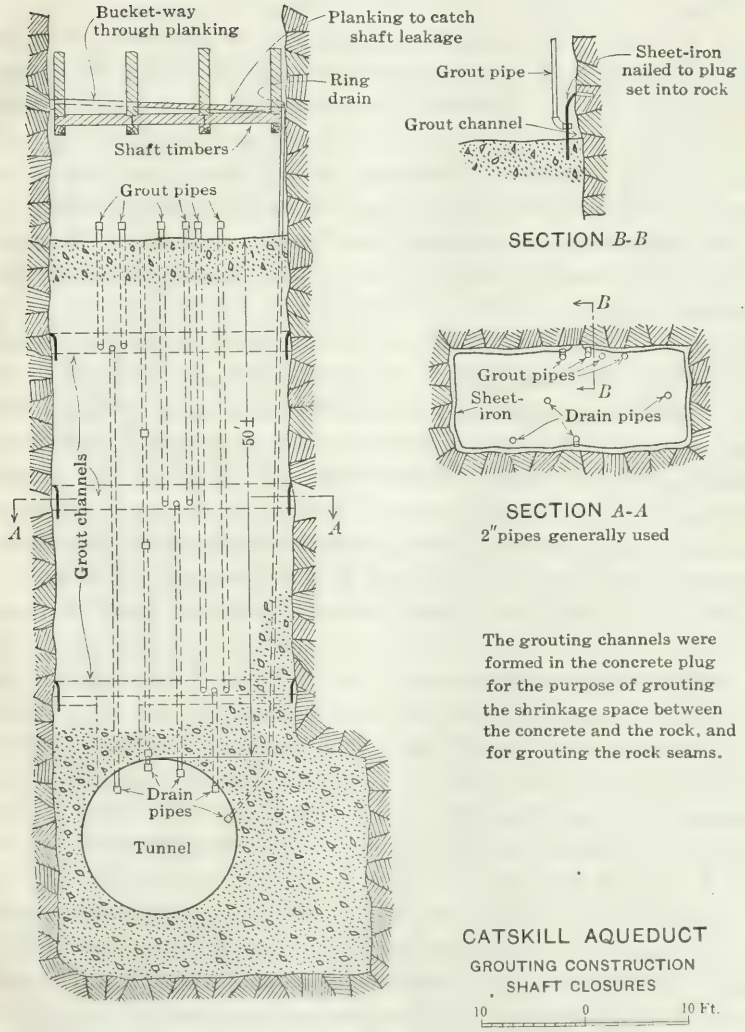

Bucket-way through planking
Planking to catch shaft leakage
Ring drain
Shaft timbers
Grout pipes
Grout channels
50 ft.
A
A
Drain pipes
Tunnel
Grout pipe
Sheet-iron nailed to plug set into rock
Grout channel
SECTION B-B
B
Grout pipes
B
Sheet-iron
Drain pipes
SECTION A-A
2" pipes generally used
The grouting channels were formed in the concrete plug for the purpose of grouting the shrinkage space between the concrete and the rock, and for grouting the rock seams.
CATSKILL AQUEDUCT
GROUTING CONSTRUCTION
SHAFT CLOSURES
10 0 10 Ft.

FIG. 10.

of the excavation down stream from the cut-off trench. A model was made, showing the position of the grout holes and the water-bearing seams, and was found to be useful in connection with the subsequent grouting.

A 3-ft. length of 2-in., wrought-iron pipe, the lower end of which was chipped with a chisel and wrapped with jute, was driven into the tops of the holes until the jute became firmly wedged and made a tight joint. The pipe was then wedged fast at the top with two small iron wedges, and the annular space between it and the sides of the drill hole was filled with grout, thus sealing the pipe in the hole. If water was flowing from the drill hole, the iron wedges and the grout were omitted, and dry cement was rammed into the top of the hole outside the pipe.

Sections of pipe of convenient length were successively screwed to the tops of the grout pipes so as to keep them always above the top of the cyclopean masonry as the latter progressed, until the holes were grouted. After the surface of the rock was covered by a sufficient weight of concrete masonry to overcome the possibility of lifting or blowing out by hydrostatic pressure, the tops of the grout pipes were connected to a Cockburn grouting machine of about 4 cu. ft. capacity, and grout was forced into the hole by an air pressure of from 25 to 90 lb. per sq. in. The grout generally used was mixed in equal volumes of neat cement and water.

The grouting extended over a period of about one year and was done at any convenient time after the concrete had attained a height of 10 ft. or more. The 45 holes extending to the lower seam took a total of 251 cu. ft. of grout, of which it was estimated that 175 cu. ft. entered the seam. The 172 holes drilled to the upper seam received a total of 1 118 cu. ft. of grout, of which it was estimated that 925 cu. ft. entered the seam.

The quantity of grout forced into any hole varied from 0 to 198 cu. ft., and only about 10% of the total number of holes took an appreciable quantity. The maximum distance through which grout was forced, making its appearance in other pipes, was 70 ft.

A number of experiments were made in connection with the grouting, which are of interest. Grout was forced under a pressure of 25 lb. per sq. in. into a hole drilled about 15 ft. back from the edge of the cut-off trench, and appeared on the face of the trench from a

horizontal seam about 6 in. below the top. Several months later, the rock covering this seam was barred and wedged off, and the seam appeared to be well filled with cement which was set hard.

Grout was also forced into a drill hole about 6 ft. back from the face of the cut-off trench, through two seams to the face of the trench. Two days later the rock was barred off, exposing the seams which were found to be well filled for a distance of 4 ft. in every direction from the hole, and partly filled 2 ft. farther out.

Grouting at Woodstock and Glenford Dikes.—These two structures close the gaps between the hills forming the Ashokan Reservoir. The dikes consist of rolled embankments with center concrete core-walls which, with the exception of short portions, were carried down to ledge rock. In the vicinity of the dikes, the rock is overlaid with a cover of clay and gravel, varying from 1 to 30 ft. in depth.

As the East Basin of the reservoir filled, it was observed that some seepage was taking place underneath the dikes, which increased as the water level in the reservoir rose. Although the loss of water was not serious, it was considered advisable to take measures to stop the seepage and to improve the character of the foundations. To determine to what extent this might be accomplished, it was decided to grout a portion of the foundation of the Woodstock Dike.

Operations were limited to a length of 379 ft., the total length of the dike being approximately 2 500 ft. Holes were drilled, spaced about 12½ ft. apart, on the center line of the dike where the core-wall extended to rock, and on a 5 or 6-ft. off-set line up stream from the center line, where the core-wall was not carried down to rock. Thirty-five holes were drilled, their total depth through the embankment and earth cover aggregating 426 lin. ft., while the depth of the core-borings totaled 717 lin. ft.

Where the holes were on the center line of the dike, a 4-in. casing was driven to the core-wall, a distance of approximately 9 ft. A 3-in. hole was then drilled into the core-wall to a depth of 2 ft., into which a 2½-in. casing was driven and grouted, giving a tight seal. The 4-in. casing was then withdrawn. For the holes up stream from the core-wall, a 2½-in. casing was driven to rock. The rock was chopped out to a depth of from 4 to 24 in. and the casing was driven tightly into it.

The core-borings were made with an **A** bit, which cut a hole of approximately 1¾-in. diameter. The rock encountered was bluestone,

varying from a coarse stone of good quality to a very fine grained stone. In general, the holes averaged 40 ft. in depth, 20 ft. of this being in rock. Tests made by pumping water into the holes showed loss of water at practically all those drilled in rock, the maximum quantity that could be pumped into a hole without accumulating pressure, having been 66 gal. per min.

On the completion of the drilling, the holes were grouted. Two Cockburn-Barrow grout machines were used to mix the grout, discharging into a wooden box, from which it was pumped by a Douglas hand grout pump into the hole to be grouted. The mix varied in consistency from ½ bag of cement and 40 gal. of water to 5 bags of cement and 40 gal. of water. The general method of procedure was to attach the grout pump to a hole and continue pumping in grout as long as the hole would take it. In the majority of cases, the grout did not show in adjacent holes. In one instance, however, it appeared in a hole 37 ft. distant from the one grouted.

On the completion of the grouting of the rock, the casing was first pulled about 12 to 18 in. and grout was then pumped in, keeping the pressure at a maximum of 7 lb., except that after work had been done on a hole and then resumed after an interval, a slightly higher pressure was used for a short time. When the hole would take no more grout, the casing was pulled another 18 in., and the grout applied once more to refusal. This procedure was continued, alternately pulling casing and grouting, until the casing had been raised so that the bottom was at the elevation of the flow line.

The maximum quantity of grout forced into a hole in the rock was 1 078 bags of cement and the minimum 13 bags. The maximum quantity forced into a hole in the earth cover above the rock was 496 bags and the minimum 20 bags. A total of 1 881 bbl. of cement was used in this experimental grouting. It resulted in an appreciable reduction in the amount of seepage for the portion of the dike treated. Measurements made soon after the completion of the grouting, indicated a reduction of approximately 50% in the amount of seepage.

In view of the satisfactory results obtained in grouting the experimental section, it was decided to grout the foundations of the Woodstock and Glenford Dikes for their entire lengths, except a few sections which it was thought unnecessary to treat.

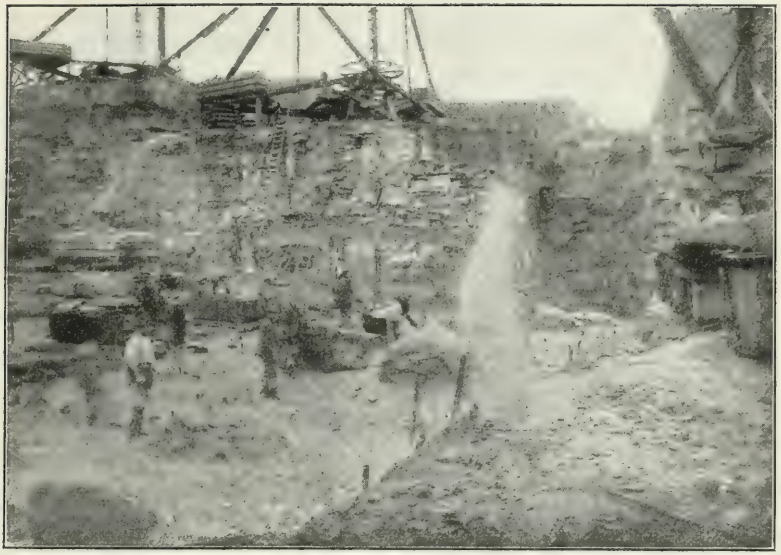


Fig. 11.—Grouting Seam in Rock, 60 Ft. Below the Creek Bed, Under Olive Bridge Dam.

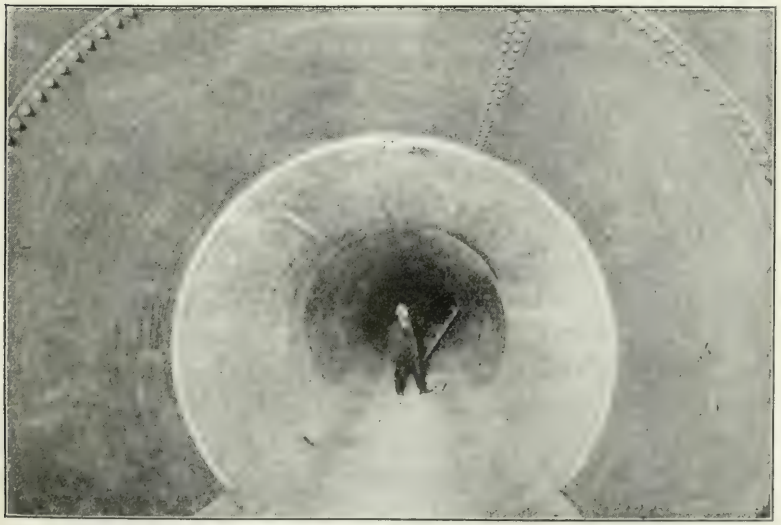


Fig. 12.—Mortar Lining in Steel Pipe Siphon, 2 In. Thick, Placed by Pouring Grout from Surface.

A modification was made in the procedure originally followed. Where the core-wall was founded on rock, the holes were drilled in two rows, 2 ft. apart, spaced about 20 ft., center to center, in each row, the holes being staggered. In the Woodstock Dike, the holes were drilled through the core-wall and into the rock, the method of sinking and sealing the casing being the same as that described in the experimental work. At the Glenford Dike, it was impossible to drill holes through the core-wall because of the location of the Ulster and Delaware Railroad on that dike; the casings, therefore, were sunk up stream from the core-wall, with the same spacing as described previously.

For the portions of the Woodstock Dike where the core-wall was not carried to rock, holes were placed on offsets and spaced 5 ft. apart; 4-in. casing was sunk to rock, and, at alternate holes, 2½-in. casing was placed inside the 4-in. casing and securely sealed into the rock. The holes through the core-wall and into the rock were $2\frac{1}{16}$ in. in diameter instead of 1¾ in., as in the experimental work, and were drilled to a minimum depth of 20 ft. into the rock. The total linear feet of holes drilled at Woodstock Dike, including the experimental section, was 11 182, and at Glenford Dike, 9 190.

The same general procedure was followed in grouting the holes as in the experimental grouting, but some changes were made. In order to economize in the labor incidental to the use of hand-pumping of grout, water pressure supplied by steam pumps was substituted for the Douglas hand pump, four grouting units being in operation at all times, with piping extending the full length of both dikes. As before, Cockburn-Barrow grout machines were used for mixing the grout, wooden troughs conveying it to hollow iron cylindrical pressure tanks.

These grout tanks consisted of 12-in. iron pipes, 4 ft. long, laid horizontally, two of which were mounted on a portable wooden truck. Suitable 2-in. pipes and fittings made it possible to use each tank alone or in sequence. Pressure, when required, was applied at one end by connection with the water line, and the grout in the tank was discharged by displacing it with water. This method was found to work very smoothly, resulting in a saving of time and more effective grouting on account of the continuous operation.

The grouting of the earth cover and embankment above the rock, at the Woodstock Dike, was completed before the rock was grouted. The row of 4-in. casing without the inner 2½-in. pipe, was grouted first, and then the other row, the grouting of cover to flow-line elevation being accomplished by raising the 4-in. casing as desired. After this cover grouting was accomplished, the 2½-in. casing was used to grout the rock. A total of 7 564 bbl. of cement was used for grouting at Woodstock Dike, including the experimental section, and 5 613 bbl. at Glenford Dike.

The results of the grouting at both dikes have been very satisfactory, the estimated reduction in seepage, with a full reservoir, being 75% at Woodstock Dike and 90% at Glenford Dike. In addition, the foundations under the dikes have been improved materially, due to the consolidation of the disintegrated rock and the filling up of the seams and cavities.

Grouting the Foundation Under Kensico Dam.—Kensico Dam is a gravity structure of cyclopean concrete masonry, with a maximum height of 310 ft. above the bottom of the excavation. The dam is across the Bronx River, between parallel ridges which rise to a height of about 180 ft. above the bed of the river. At the site of the dam, the quality and attitude of the rock was developed by core-borings which clearly revealed the geologic structure of the valley floor.

The east ridge is composed of gneiss, the west ridge of schist, and between the two hills lies a bed of crystalline limestone about 400 ft. thick. The excavation for the dam was carried down to generally sound rock. The limestone was found to be deeply decayed in places, especially along the contact with the gneiss, where the decay follows the contact for a depth of more than 100 ft. below the general level of the rock floor and affects the adjacent gneiss. No decay was found at the contact between the limestone and the schist. Both the gneiss and the schist showed less depth of decay than the limestone but, in places, these rocks were shattered and showed the effect of crush zones by lines of weakness along the bedding planes.

The foundation of the dam required more or less extensive grouting at three points, one in the schist near the west end of the dam, one at a soft seam in the limestone, and the third covering the entire east side of the gorge.

At the west end of the dam beyond the cut-off trench, the mica schist was found to contain occasional cross-bedding seams, and the area was explored by a few drill holes, 25 ft. deep, which were tested with water under pressure. Some intercommunication was found in these holes, although holes drilled in the bottom of the cut-off trench just east of this area were tight. The area was treated by drilling two lines of holes 11 ft. apart, with a 3-ft. spacing in each line, and 25 ft. deep, for a length of 40 ft. along the dam. After the masonry had been placed to a sufficient height, the holes were filled with grout under pressure.

At one place a crush zone in the limestone crossed the foundation. At the surface of the rock it was about 15 ft. wide, but it narrowed with depth. It was followed down by the cut-off excavation until it was less than 3 ft. wide, and as it was entirely filled with compact material, further excavation was deemed to be unnecessary. Holes 25 ft. deep were drilled over the area, and on testing with water and uranium, those to the west of the seam were observed to be more or less in communication with each other. When the masonry in the dam had been placed, providing a cover of 35 ft. or more in thickness, the holes were grouted.

On the east side of the gorge, where a disintegrated zone was found at the contact between the gneiss and the limestone, more extensive grouting was necessary than elsewhere. The rock lay in beds about 2 ft. thick, and when exposed the bedding planes opened up. Several layers were excavated until fairly sound rock was reached but, under exposure to the air and summer heat, a further opening of the bedding planes took place. The excavation of the cut-off trench, which was carried down to a depth of 40 ft. below the general level of the excavation, had shown that several seams in the rock were open and water-bearing.

Holes, 25 ft. deep and about 25 ft. apart, were drilled normal to the inclination of the strata, and 2-in. pipes were fitted into them and extended up through the masonry. Two rows of holes were also drilled in the bottom of the cut-off trench near the sides, about 15 ft. apart, longitudinally. The tops of the lower holes in the gorge were at about Elevation 65, and as they were not grouted until the concrete had reached Elevation 192, the grout pipes were in some cases 125 ft. in length.

Pressure tests, using water colored with uranium, were made on each hole to determine the influence on the other holes. As a result of the tests, it was concluded that the cut-off trench effectively intercepted the more open seams, and that the grout pipes could be divided into three groups: First, the holes up stream from the cut-off trench, where they all appeared to connect freely with each other; second, the holes in the cut-off trench which evidently penetrated a region not reached by the others; and, third, all the holes down stream from the cut-off trench, which were evidently quite freely intercommunicative.

In grouting, each of the three groups was treated separately. Usually, the pipe which took water most freely was the first to be grouted. As the grout appeared in other pipes, they were capped, and the pressure continued until the hole refused to take more grout. Then the pipe was capped, and connection was made to another pipe, the uppermost pipe being left open for a vent. Eventually, all the pipes were filled with grout, capped, and left embedded in the dam masonry. With the reservoir full for two years, there has been no indication of any leakage on the down-stream side of the dam.

Miscellaneous Grouting.

Mortar Lining Inside of Large Steel Pipes.—Steel pipe siphons were used for crossing valleys along the line of the aqueduct where the rock was not sound or where, for other reasons, pressure tunnels would be impracticable. The steel pipes vary from 9 ft. 6 in. to 11 ft. 2 in. in diameter, and are lined with 2 in. of cement mortar, enveloped with concrete, and covered with an earth embankment.

The mortar lining was placed in two operations (see Fig. 12): First, an invert about 8 ft. wide was placed in the bottom of the pipe, using a 1:2 mortar and finishing with a screed board and trowel; and, second, the portion above the invert was lined by using a wooden form built up of curved panels inside the pipe, and pouring grout from the surface of the ground through vertical 2-in. pipes screwed to the top of the pipe at 15-ft. intervals, one pipe being used to grout a 15-ft. section and the next for a vent.

The grout was mixed by hand on the surface in boxes arranged so that one batch would be mixed while the other was discharged. The grout was poured through the pipe screwed into the lower hole, until it ran out of the 4-ft. riser inserted in the upper hole. The pouring

was continued for some time to allow the air to escape. This method permitted the discharge of the surplus water and laitance, and effectively took care of any shrinkage. The grout for the portion between the invert and the springing line was mixed 1 : 1, while, for the portion above the springing line, a 1:2 mixture was used.

In one of the siphons, the mortar lining was placed by the cement gun, without the use of forms. The apparatus consisted of a double tank with air-tight bulkheads and a revolving feed for sand and cement. The mixture was discharged by air under 40 lb. pressure through a rubber hose. As the mixture reached the nozzle, it came in contact with water discharged through another hose, and the combined spray was discharged with great force against the inside surface of the steel pipe. Although the results obtained with the cement gun were satisfactory, it was not well adapted for the work and was not used subsequently in the other siphons.

Grouting Expansion Joints in Cut-and-Cover Aqueduct.—The concrete in the arch of the cut-and-cover aqueduct was placed in lengths of 30, 45, 60, and 75 ft. The vertical expansion joints at the ends of the arch sections were formed either by tongue-and-groove joints cast in the concrete, or by steel plates. Owing to temperature changes, these joints opened up in cold weather, the width of the openings varying from about $\frac{1}{32}$ to $\frac{1}{4}$ in., depending on the temperature of the air and on the length of the arch sections. Hydrostatic tests, made by filling portions of the aqueduct, showed that neither the tongue-and-groove nor the steel-plate expansion joints were tight against outward leakage in all cases. After some experiments and tests, it was decided that the most effective method of making the joints tight was to grout them with cement. Accordingly, the joints in several miles of aqueduct were treated by grouting them in the winter months when they are open the widest. Only the joints that were appreciably open were grouted.

The following method of grouting was found to be successful on one section of the work. The grout from a 2-qt. tin coffee-pot was poured into a small tin funnel and through a $\frac{1}{2}$-in. rubber tubing, about 12 in. long, into the joint. The joints were usually raked out in advance of the grouting, using a 12-in. pruning saw having very coarse teeth on one edge. The grout was poured near the top of the

joint, from the inside of the aqueduct, the leakage of grout into the aqueduct being stopped by caulking the joint, usually with oakum.

Where the joints were wet, on account of inward leakage due to ground-water, it was necessary to grout them by pressure, using a Douglas hand pump for this purpose. They were first drilled by hand at convenient points, and into the holes were inserted $\frac{3}{4}$-in. pipe nipples, 6 in. long, threaded at one end. The nipples were caulked into the joints with lead wool, and the inside edges of the joints were also caulked with the same material.

The mix generally used for grout poured into place was 5 parts of cement to 8 parts of water by volume. This consistency was found to be a little too thick for the hand pump and, for the wet joints, a mix consisting of 3 parts of cement to 8 parts of water was used. The grouting was very effective in making the joints tight against leakage.

Grouting Caving Ground in the Grade Tunnel.—In driving one of the headings of the Reynolds Hill Tunnel, several slips occurred, and, in a 15-ft. stretch, the timbering was badly distorted by the slide. In order to excavate safely the bench through this section, the space outside the timbering which had previously been dry packed, was grouted. Concrete was first placed back of the timbering for a height of about 4 ft. above the wall-plates and, at each end of the section, concrete cut-off walls were built as high as possible between the lagging and the rock, the closures being made with brick masonry. The dry-packed space over the arch between the cut-off walls was then grouted by a tank grouting machine. More than 30 cu. yd. of 1:1 grout were placed. No trouble was experienced subsequently in excavating the bench through this section.

Grouting in Tunnel Where Concrete Lining Had Disintegrated.—Some time after it was placed, the concrete lining in a portion of the Eastview Grade Tunnel was found to be undergoing a progressive slow disintegration, caused by the infiltration of the surrounding ground-water which contained certain chemical elements injurious to concrete. The portion of the tunnel affected, a stretch of about 1 700 ft., was relined with brick 12 in. thick.

To take care of the water seeping through the concrete lining, sheets of steel were used throughout the side-walls and arch of the lined section, and kept uniformly away from the concrete face by

ordinary spruce plaster lathe, to provide drainage and grouting space. Before the brick lining was placed, a bituminous water-proofing fabric was deposited on the sheet metal water-stop and on the invert concrete.

Along the center line of the invert, in the lined section, a 4-in. tile pipe was laid with open joints to receive the drainage water. Transverse drainage grooves cut in the invert conveyed the water from behind the metal stop to the center line drain. This drain was provided with 2-in. pipes, 12 in. long, extending up through the brick lining on the invert. Before grouting the space behind the water-stop, this drain and the connecting grooves were pumped full of grout, connections being made to the 2-in. pipes in the invert.

For grouting between the metal stop and the original concrete lining, 2-in. pipes were set, extending through the brick work at three levels on both sides of the tunnel. The grouting was done in successive upward stages, grout being supplied to the lower middle and upper level of pipes until it appeared at the next higher level, each stage covering the entire length of lined section before the next stage was begun. A Douglas hand pump was used to place the 1 : 1 grout which was mixed in boxes carried on flat cars and stirred continually while being used. On account of the hydrostatic pressure behind the brick lining, the side-walls were braced throughout the length of the lining during grouting, with 4 by 6-in. struts, spaced 5 ft. on centers, and wedged in place.

In addition to relining a portion of the tunnel with brick, additional precautions against the disintegrating effect of the ground-water were taken by grouting the space above the arch for the entire length of the tunnel, about 1 mile. This space in the roof between the concrete and the rock had been partly dry packed at the time the concrete lining was originally placed.

The grout pipes were 1½ in. in diameter, provided with couplings at their ends, and placed in holes drilled through the concrete. A pair of pipes, one a grout and the other a vent pipe, was placed through the arch concrete at intervals of about 50 ft. The inner ends of all pipes were wrapped smoothly with cotton cloth in order to make them fit tightly in the holes, and were driven in place by a sledge-hammer.

The grout, consisting of 1 bag of cement, 1 bag of sand, and usually 12 gal. of water, was placed by means of a Canniff tank, the

grout being forced in to refusal under about 90 lb. pressure. The quantity of grout placed averaged ½ cu. yd. per lin. ft. of tunnel.

General Conclusions in Regard to Grouting.

The experience in the grouting operations in connection with the Catskill Water-Works, showed that grouting is very effective for backing up, and filling the spaces behind, tunnel linings. A mixture consisting of 1 bag of cement, 1 bag of sand, and from 6 to 8 gal. of water, was found to be well adapted for this purpose. The air pressure should not be greater than is necessary to force the grout into place. The pressures actually used varied from 30 to 90 lb. per sq. in. To complete the filling of the voids between the tunnel lining and the rock, particularly in the high places in the roof adjacent to the "vent" pipes, air pressures up to 300 lb. per sq. in. and neat cement grout were used.

Good results were obtained in cutting off the leakage of water into the tunnels by grouting the water-bearing seams. For this purpose, it is essential to control the inflowing water by collecting it behind drip-pans or a steel shell, and draining it by pipes through which the grout is subsequently forced into the seams. Neat cement grout, about 6 gal. of water to 1 bag of cement, was found to be effective for this purpose. For fine openings a very thin mix must be used in order to obtain good results. The pressures should be sufficient to overcome the ground-water head and force the grout into the rock.

In sinking wet shafts, grouting was successful in cutting off leakage. The best results were obtained by alternately drilling and grouting, using holes about 10 to 15 ft. deep in the bottom of the shaft. The consistency of the grout should be adjusted to the size of the seams. In some cases, a little addition of bran, oats, etc., to the neat cement grout will aid in plugging the crevices in the rock.

The results obtained in treating the foundations of dams and in cutting off leakage underneath the dikes by drilling holes and grouting the rock seams under the dam structures were satisfactory. It is desirable in such cases to explore the rock under the dams thoroughly by diamond drill borings for the purpose of locating weak zones and open seams. The location of the holes and method of grouting will depend on the natural conditions encountered.

In conclusion, it should be stated that the procedure to be adopted in any grouting operation must take into account all the factors that enter into the problem. No one method is applicable to all cases. The conditions encountered during the grouting may make it advisable to modify the procedure materially. Good judgment and skillful adaptation to meet varying conditions are essential to a successful conclusion.

This paper was prepared at the suggestion of the Chief Engineer, J. Waldo Smith, M. Am. Soc. C. E., to whom the writers are indebted for advice and many valuable suggestions. Many of the data were obtained from reports and contributions by members of the staff of the Board of Water Supply of New York City, and the writers wish to express their appreciation of the assistance given them, particularly by A. D. Flinn, Robert Ridgway, G. G. Honness and M. H. Freeman, Members, Am. Soc. C. E., L. White, Assoc. M. Am. Soc. C. E., and Mr. L. B. Stebbins.

DISCUSSION

Mr. Ridgway.

ROBERT RIDGWAY,* M. AM. SOC. C. E.—The speaker wishes to express his appreciation of the great value of this paper to constructing engineers. Not only is the subject one of much importance, but the authors are especially qualified to present the paper because of their first-hand knowledge of the operations which they describe. Its value is enhanced by the fact that it was prepared while the work was still fresh in mind, in consultation with those familiar with all the details of the operations and with all the records available. Many lessons learned at great cost on works of construction are lost to those who take up similar work at a later period because the organization that learned and applied them is broken up on completion of the work, leaving behind only what a recent writer has described as "dead and fragmentary records" which are practically lost when they are filed away in some vault for safe keeping. The construction of large water supply systems like those of the Catskill project is not a continuous performance. Sometimes there are intervals of many years between the completion of one project and the beginning of another, and the earlier work is largely forgotten when the later one is undertaken. If a record of the unusual features of design and construction is not left behind in some such form as the paper under discussion, those who take up similar studies at some future time must go over the same ground and inevitably make some of the same mistakes as their predecessors.

The grouting methods of the Catskill work were highly developed, and were successful as a result of a willingness to experiment in practical ways and of the large experience gained on a work of such magnitude and varying conditions. It is extremely fortunate, therefore, that this interesting paper has been prepared describing the methods adopted, and it is hoped that the Chief Engineer will suggest to his staff the preparation of other equally valuable papers descriptive of such features of the work, for example as the deep pressure tunnels for which there was little precedent in earlier projects.

The speaker's first experience with grouting was in the grade tunnels of the so-called New Croton Aqueduct during the years 1888 to 1890. This tunnel was lined with brick backed with rubble masonry. The brickwork and rubble were laid at first in natural cement mortar, but in the later stages of the work Portland cement mortar was used. On account of the unfortunate conditions then prevailing, for which the engineers were in no way responsible, it became necessary to repair long stretches of defective masonry lining. It was the practice of some of the contractors to omit most of the mortar in the rubble masonry and in some cases to omit the rubble masonry altogether. The scamp work

* New York City.

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was done in such a wholesale manner that it was necessary to organize for the repairs in a comprehensive way. Where the brickwork was defective and where large voids existed in the backing, it was of course necessary to tear out and rebuild. Where it was a case of supplying the mortar in the rubble backing it was decided, after considerable discussion and experimenting, to resort to grouting methods.

An experimental period had to be passed through at first. Holes were drilled through the brick side-walls and a thick mixture of grout was poured into them by means of funnels. This method, of course, could not be adopted in the upper portions of the arch, nor could any pressure be applied to the grouting. Besides, it was a retail way of doing a wholesale job and was soon discarded.

The method later adopted, and which proved to be successful for the work in hand, required the use of hand-operated Douglas pumps. A 1 : 1 mixture of natural cement and sand was mixed by hand in open boxes and pumped through holes drilled in the brickwork. Natural cement, or as it was then called, "American" cement, was used because much of the masonry was specified to be laid with that cement. In order to insure practically continuous pumping two boxes were used, the mixing going on in one while the charge was being pumped from the other. A set of nine holes was drilled every 10 ft. in the side-walls and the arch. The metal nozzle of the discharge hose, wrapped with cloth or burlap, was driven into these holes and the grout was pumped in to refusal. Generally, four men were used at the handles of the pumps, the number being increased to six when the pumping was hard. The lower holes on each side were grouted first and the key holes last. The grout was driven long distances, sometimes several hundred feet, and frequently was lifted over the arch from the lower holes in the side-walls. Grouting in one hole frequently blocked many others, but the latter were subsequently tried and sometimes took an appreciable amount of grout.

Prior and subsequent to the first grouting the masonry was carefully sounded with steel rods for indications of cavities or other defects. Those engaged on such work soon became very expert and could tell in a general way from the tone given out by the rod whether or not the brickwork and its backing were defective. Where the soundings or the drill holes, together with the cross-sections, indicated the existence of large cavities, or where the brickwork was defective, test holes were cut and the defective work removed and rebuilt as far as necessary before any grouting was done in the vicinity. After the first grouting, another set of holes was drilled and grouted where soundings indicated that further work was necessary. Again the work was sounded, and a third grouting was done in the limited number of places requiring it.

In the report of the Aqueduct Commissioners for 1887 to 1895 is a general description of this grouting by the Chief Engineer, and a state-

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ment is made by him that 384 700 bbl. of cement in all were used for grouting, exclusive of the cement used for the reconstruction of defective or missing masonry by hand. Of this amount probably 15% was wasted through leakage and other causes. As the purpose of the work was to fill voids in the masonry backing, no attempt was made to grout seams in the rock. The methods were crude as compared with those used on the Catskill work, but accomplished the results desired. Although the voids were almost completely filled with grout it appeared from the speaker's observations that the latter did not set up well. Samples taken out by him a year or so afterward showed the natural cement to be of chalky consistency, with little strength. Apparently, natural cement is not suitable for grouting work except in cases where its function is merely to fill voids. Where strength and density are essential requirements only Portland cement should be used.

Grout was used extensively and successfully in the construction of the tunnels of the subway system of New York City. This grouting was important, but generally speaking it was not of such difficult character as that of the Catskill work. In earth and rock tunnels, grouting was done to fill the voids which always exist over the concrete arch and to reduce leakage. It was done in the soft ground shield-driven tunnels adjacent to the East River to prevent settlement of ground overhead, and where the work was done in compressed air under and in the vicinity of the East River to reduce the loss of air from the tunnels. The methods have been quite successful in accomplishing these objects. The pressures used have not been high, corresponding to those used in the "low-pressure" grouting described by the authors.

Within the past year, grouting was successfully used to reduce the leakage in the Canal Street Station of the New York Municipal Railway subway from considerably more than 100 gal. per min. to less than 2 gal. per min. The work was of a most difficult nature and the excellent results were accomplished only because the engineers in direct charge were possessed of an intimate knowledge of modern grouting methods and their limitations. The speaker will not go into the details, as he understands that they have been covered in the written discussion by A. J. Mayell, Assoc. M. Am. Soc. C. E., who was the Assistant Division Engineer on the work. The guidance of M. H. Freeman, M. Am. Soc. C. E., formerly of the Engineering Staff of the Board of Water Supply, was invaluable on this work.

It is apparent from the authors' description that the concrete of the tunnel and shaft lining is of excellent quality, due to the great care taken in placing it. Otherwise, the speaker is convinced that the grouting would not have been a success. To accomplish the results described it was essential that the concrete be strong and dense. Grouting will not change poor concrete into good concrete and it adds little or nothing

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to the strength of poor concrete. To obtain good concrete it is necessary not only to mix it properly, but to keep all water pressure away from it while setting. It will be porous unless this is done, as many engineers have found when they have not taken proper precautions.

Grouting is expensive, and to secure the best results at the least cost and in the quickest time it is wise to employ only those experienced in such work. To start with an untrained force usually results in discouragement and sometimes the abandonment of the work. After the men are well broken in they seem to develop an instinct for doing the work in the right way in the shortest time and accomplish what before seemed to be impossible.

If the grouting work under a contract is to be done as carefully and as thoroughly as it was on the Catskill work it is necessary to draw the specifications with care and to make provision, as was done on that work, to pay the contractor for each step in the operations. For example, the cost to the contractor of moving the apparatus and of making connections to the grout pipes is considerable and sometimes many connections are made to pipes which take no grout. If payment is made merely on the basis of grout pumped in, this would work a hardship on the contractor who would receive no payment for the work done on those holes into which no grout was forced. In the case of the Catskill work payment was made under unit prices for each connection effected.

The authors state, referring to the cement, "experiments show that the finer the grinding, the greater the loss in the strength of the grout on account of the more thorough hydration of the cement. The coarse particles impart strength to the cement, and, for that reason, very fine grinding may not be desirable." This is interesting, and contrary to the idea of the speaker, who had been of the opinion that finely ground cement would be better for this and for all other purposes. Where sand is used it must be very fine, as the authors point out, to prevent segregation as well as clogging of the holes.

In the discussion preceding the preparation of the Catskill tunnel contracts the speaker favored the cut-off walls mentioned by the authors. It was believed that by confining the grouting to relatively short stretches of tunnel and completing each stretch before passing to another the cement and sand would not segregate and that better results would be obtained. Experience, however, showed that they did not work out well in practice and hence their use was discontinued.

The authors point out the care necessary to prevent a blast of air following the charge of grout where the tank machines are used. Where small spaces are to be filled completely and the amount of grout, consequently, is small, it has been found advisable in many cases to use a hand pump, thus avoiding the objection of trapping in the cavity air which prevents complete filling of the hole.

Mr. Ridgway. It has been assumed by many that grouting could be used to solidify unstable sand foundations, particularly where the sand is fine and saturated. The speaker has seen a number of unsuccessful attempts made to do this. Perhaps with infinite pains and at great expense something could be accomplished in this direction, but he does not believe it to be a practical proposition.

As of interest in connection with the treatment of dam foundations, attention is called to the paper by the late Charles S. Gowen, M. Am. Soc. C. E., entitled "The Foundations of the New Croton Dam",* wherein is described the novel method used to fill with clay the voids existing in a limestone cave under the dam. Plastic clay in the form of "sausages" or cylinders was fed into a casing pipe which had been carried up from the cavity 100 ft. or so and bedded in the masonry of the dam. The clay was driven by means of a pile-driver. The details are well described in Mr. Gowen's paper, and it is stated that a total of 66 951 lb. of clay was driven, apparently completely filling the cavity, as evidenced by the rising of the clay in a tell-tale pipe outside of the dam structure. In the speaker's opinion the method thus described could be used in many cases where the problem is merely to fill voids with a water-tight material and where it is not necessary to use material that will solidify like cement.

Mr. Smith. J. Waldo Smith,† M. Am. Soc. C. E.—This paper is a creditable record of many years of work under greatly varying conditions. The widely diversified aqueduct work provided an opportunity for the development of new ideas, and the 35 miles of pressure tunnel offered a much better opportunity than has before presented itself for the further development and perfecting of grouting methods. Due to the great number of men engaged on it and the variety of ideas that were presented, it is safe to say that more has been done to develop and put on record methods that will be useful in the future to those undertaking such work than ever before.

It is part of every one's duty to place in the archives of the Society records of methods that have been developed, for the benefit of others coming after or which will help the wayfarer looking for assistance along the road. Mr. Ridgway has stated that he hoped the speaker would suggest other papers. He has done so; but due to the war and perhaps also to the fact (which should be a matter for congratulation) that in most cases the men best suited to write these papers have gone on to better positions before their work was finished and before they had time to do so, the proposed papers have not been prepared.

The speaker still hopes that some of the men, although they are no longer connected with the work, will undertake to present a series of papers which will cover all the special features of the Catskill Aqueduct.

* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 469.

† New York City.

Mr. Smith.

The one that he wants to see more than any other is a comprehensive paper on the pressure tunnels, because the pressure tunnels, the grouting of which is so fully described in this paper, were among the principal features of this work.

If the speaker had undertaken to have a number of the engineers submit the data and had himself prepared this paper, he is sure he would have been very unhappy. He could not have done it as well as they have, and it is much better for those who have been close to the work and who have really done the work, to obtain whatever glory or credit or publicity may come to them through the presentation of such a paper as this.

The authors have said that this paper was prepared at the speaker's suggestion. Well, such suggestions are part of his religion. He believes that a man in the position he has been in owes it to his associates (there have been no assistants on this job) to see that they have a chance to develop, and to help them onward and upward. It is the duty of all the members of the Society who are in executive positions to do what they can to develop the younger men, to allow them to accept responsibility, or to force it on them. To place such responsibility brings results which would be beneficial to any project.

Mr. White.

Lazarus White,* Assoc. M. Am. Soc. C. E.—This paper is so thorough and covers the subject so well that it is difficult to enlarge upon it. There is one impression which the authors did not intend to convey, but might be obtained due to the great emphasis on the grouting operations; that is, that the water-tightness of the tunnels was due almost entirely to the grouting, and that the concrete tunnel lining itself is of minor importance. This, in the opinion of the speaker, would be unfortunate, for, as Mr. Ridgway remarked, the tunnel lining itself is of primary importance and without a thoroughly good lining it would be hopeless to secure water-tightness for effective grouting. Those who examined the tunnels will bear witness to the fact that the lining was first-class with very few visible defects. To get good lining, every care was taken in leading the water off with drip-pans and grout pipes, as the authors describe. This prevented damage to the lining, allowing the concrete to set while it was entirely relieved of water pressure and water percolation. The same drip-pans and grout pipes which primarily served to lead off the water and protect the lining were the instruments for the subsequent grouting.

Had the vent pipes been blocked, the lining itself would be found to be very tight without the grouting. In one instance this was done, and the lining withstood a pressure of 135 lb. per sq. in. without leakage. This was described by the authors. The grouting, in the opinion of the speaker, was primarily designed to back up the lining, and to transfer the internal pressure through the concrete and grout to the rock backing.

* New York City.

Mr. White. It is the rock in pressure tunnels which, after all, carries the pressure, and it must necessarily carry it. In cases where sections of the tunnel lining failed, it was due to the fact that the rock backing was unable to carry the heavy internal pressure. On the Rondout Tunnel there were heavy layers of limestone between which were layers of greasy clay, and when the internal pressure was applied, the strata moved, cracking the lining. These stretches were repaired by putting in steel lining capable of withstanding the internal pressure without the aid of the rock.

As the authors have stated, there were people who predicted that the lining would fail in very numerous cases, and favored the use of a continuous steel lining to take up the internal pressure independently of the rock. It was decided that this would be too expensive, and it would be cheaper to test the tunnels hydrostatically and repair the weak spots in places where that was necessary. This was done and resulted in great economy.

It is to be noted that the grouting operations at the end of the work were quite different from those at the beginning, and much simpler. An elaborate arrangement of grout pipe, drip-pans, cut-off walls, collecting pipes, etc., which were at first shown in the design, were later dispensed with. The grouting methods which were subsequently used in the aqueduct were the result of a great deal of experience and developed from many years of work.

The impression might be given from the statement by the authors on page 1037 that the grouting behind the pneumatic caissons was responsible for their tightness when the air was taken off. The sealing of these caissons was accomplished by the simple expedient of dropping them on a bench of concrete so that the cutting edges compressed a heavy wad of oakum. Immediately after this the air was withdrawn, and the caissons were very dry. The grouting subsequently done behind the caissons was in the nature of backing them up and making more permanent the sealing accomplished by the oakum.

Mr. Hogan. JOHN P. HOGAN,* M. AM. SOC. C. E.—This paper is so complete in itself that there is very little that can be added to it. The only thing that any one can do in discussing it is to attempt to emphasize or bring out more strongly some of the points covered.

Of the points that the speaker would like to emphasize is one which Mr. White also has made, that in order to be successful grouting must be simple. Complicated systems of connecting pipes and elbows, leading a large quantity of water to a given point, and attempts to grout a large area during one operation, never succeed. Straight pipes and small drip-pans are needed. A small drip-pan is necessary for two reasons: first, in order that it may be grouted successfully, and,

* New York City.

second, to avoid bringing too large an area of the tunnel lining under pressure. The latter requirement became very important in high-pressure grouting, particularly in the Hudson River Siphon, where a grouting pressure of 700 lb. per sq. in. was used. In some cases on that work where the lining ruptured, there was evidence that the drip-pans were too large, and too large areas of the tunnel lining were exposed to the high-pressure grouting. Mr. Hogan.

The statement is made by the authors that the purpose of low-pressure grouting was: "(*a*) primarily to fill the void spaces between the lining and the rock, particularly above the arch, and, at the same time, to form a blanket over the construction joints in the arch; (*b*) to fill the voids in the dry packing where the latter was used; and (*c*) to fill any porous or honey-combed spots in the concrete." If anybody has had any success in filling porous or honey-combed spots in concrete with grout, the speaker would like to know about it, because, for about six weeks, experiments were made in the Rondout Siphon to impregnate leaky concrete with grout. The Board provided for this purpose a special grouting pad 2 ft. square, and consistent and persistent efforts were made to use it, with a total net result of loss of time and loss of money. The attempt to grout leaky concrete with this pad was like trying to use an irresistible force against an immovable resistance.

The speaker merely mentions this because the authors have carefully avoided any further mention of it in the paper. One does not get grout in place anywhere unless one has circulation. The simplest form is the grout pipe and the vent pipe; when you get grout out of the vent pipe, you know the intermediate space is filled. In the same way in filling grout seams, wherever there was any interconnection and circulation through the seams it was possible to get grout into the seams; but, in general, very little progress was made in attempting to grout the seams in the rock unless there were such interconnections.

The provision of sufficient vent pipes for circulation and for getting rid of surplus water and air also enables you to see what you are doing and what you are accomplishing.

It has been correctly stated that the lining and not the grouting was the primary cause of the tightness of the tunnel. The important thing in placing a tight lining is to prevent water pressure on the concrete while it is setting. It is necessary therefore to lead water through the concrete to prevent the accumulation of pressure, and it is necessary in some way to fill the void which existed for the purpose of collecting water. If it were not for the sake of permanency, the pipes leading through the concrete for the purpose of collecting water might be closed with an iron cap, or even possibly with a wooden plug. The grout forced in through these pipes, however, in addition to stopping permanently the inflow and plugging the pipe, helps to back up the lining over the area which the drip-pan covers.

Mr. Hogan.

In grouting on a large scale, special provision must be made for carrying the grouting pipes through the forms, particularly when steel forms are used. This was done on the Catskill Aqueduct by threading holes through the forms, and by leaving couplings on the ends of pipes, so that connections could be made to them readily.

In grouting large spaces, such as existed in the dry packing back of the steel shell on the Rondout incline, special measures are necessary. In the incline, at the time the concreting was started, there was an inflow of 1 200 gal. per min. of water within a stretch of about 180 ft. The placing of the invert was the most difficult operation on account of the necessity of caring for the water in the bottom, while the invert concrete was setting. This was successfully accomplished by laying a longitudinal 8-in. riveted steel pipe with 6-in. Y-branches. After the invert was placed, the steel arch ribs and the steel shingle plates were erected on it. The dry packing and the placing of the concrete were carried on without any difficulty. Wherever there was a heavy flow from the rock a hole was drilled back into that seam, and a deep-seated pipe was caulked back into this hole so that any water under pressure might be led readily through the lining and allowed to flow freely during the lining process and during the grouting of the space outside of the shell.

The space to be grouted in the dry packing was very large, averaging about 18 in. in diameter, with a radius of about 11 ft. The speaker does not remember how much grout was used in that shell, but perhaps Mr. Freeman will supply that information.*

After the lining had been placed and the water led through it by means of these deep-seated pipes, or allowed to drain through the dry packing down into the steel pipe in the invert, there were really two stages to the process of grouting—filling the dry packing, and shutting off the water. The object was to have a still lake of water into which the grout could be forced under low pressure. The grouting went on for six days; and so carefully was the high-pressure water controlled through the deep-seated grout pipes that there was no flow whatsoever in this lake of water in the dry packing.

In other cases, as much sand as possible was used in grouting the large spaces because it was less subject to churning action of the air, because it made a stronger grout, and because the shrinkage was less. In general, unless grouting small seams, it is advisable to use as much sand as the mix will carry. The quantity is entirely a question of experiment, but, in general, about half sand and half cement can be used. In this case, however, for various reasons, no sand was used; but it is believed that an equally good and more economical result could have been obtained by using sand with the cement.

* Mr. Freeman stated that the total quantity of neat cement injected into the dry-packed section on the incline north of Shaft No. 4, Rondout Siphon, was 7 693 bags. As many as 1 200 bags were injected in one 8-hour shift.

Mr. Hogan.

The grouting was done from the bottom up, so that the grout was forced in with at least a 1-ft. head over the inlet pipe. If the grout had been put in at the top and allowed to settle through the water, the cement would have been washed away so that its setting would have been greatly retarded, if not entirely prevented. After the grout in the dry packing had set, the deep-seated pipes and the pipes in the invert were grouted under high pressure and the inward leakage reduced from more than 1 000 gal. per min. to less than 2 gal. per min.

In all these operations it was necessary, as has already been mentioned, to avoid excess of air and the blowing of air through the grout. In connection with other operations in the Rondout Tunnel, grout was cut out and was found to be very strong and dense, though somewhat stratified. There was not much separation of the sand from the cement, and the weight of some of the samples cut out was as high as 118 lb. per cu. ft., compared to about 145 lb. for concrete.

Certain breaks in the lining of the tunnel were repaired by means of a steel shell made with 15-in. channels. As Mr. White says, the breaks were due to adjustment in the rock. There has always been a question in the mind of the speaker whether a number of those breaks could not have been repaired by grouting operations alone, without using the steel shell. However, it might have been necessary to put the tunnel under pressure several times and grout it several times in order to accomplish that purpose. The uncertainty of this procedure was too great, because it appeared at that time that possibly the work in question might be a limiting factor in the use of the Catskill Aqueduct.

It would be interesting if some one familiar with the actual grouting operations could discuss the relative value of air-mixing and hand-mixing grout machines, and their respective uses. It is certain that there is a field for each. The Catskill Aqueduct was largely grouted by use of the air-mixing grout machine. However, in the Hudson River Tunnel, where the pressures were about 700 lb. per sq. in., ordinary plunger pumps were used with a suitable hand-mixing device attached. There are objections to the use of air-mixing grouting machines. The greatest objection, of course, is the quantity of air forced in with the charge, and the difficulty of getting rid of it.

The portion of the paper in regard to the grouting of the foundations of the Kensico and Ashokan Dams, and particularly the grouting of the foundations of the earth dikes, has introduced very interesting possibilities of improving the foundations of dams which have been constructed, and of which the foundations have proved to be faulty or are known to be faulty.

It did not seem to the speaker that the necessity for the high-pressure grouting is clear. The majority of the work was done at between 30- and 60-lb. pressure, and during this period the ground-water was allowed

Mr. Hogan. to flow through certain pipes to prevent the building up of back pressure. However, when it came to finally closing up a stretch, the active ground-water pressure in that stretch was often as high as 150 or 250 lb. per sq. in.; indeed, in the case of the Hudson River Siphon, it was as high as 500 lb. .It was surprising how quickly that pressure built up the moment the pipes were closed, and it was to overcome this ground-water pressure that high-pressure grouting was necessary.

Mr. onaldson. FRANCIS DONALDSON,* ESQ.—The speaker was with three of the Aqueduct contractors and has actually placed a lot of grout. Mr. Hogan stated, and Mr. White agreed with him, that grout did not have much to do with making a tunnel tight, but the speaker begs to differ.

The speaker worked on one job not many miles from New York City. It was dry rock, the concrete lining was excellent, and when it was finished and looked as dry as possible, it was not quite dry enough for the engineers, so that high-pressure grouting was required. For weeks and weeks a gang of men went up and down that tunnel with step ladders and jack-hammer drills plugging at every drop of water; wherever the least dripping from the roof occurred a hole was driven and grout pumped into it. When we got through with that tunnel, it was positively dusty.

The speaker was on several of the other jobs to which the authors refer and describe so well. In the Hudson Tunnel, we used grout at a higher pressure than was stated; this was necessary because the head outside the tunnel was 1 100 ft. and we had to force the water into the seams around the tunnel back into the crevices through which it came. It went back a great deal harder than it came in. We used a booster compressor up to the limit of our air pressures, and as we could not get enough pressure this way we pumped in water on top of the grout with a high-pressure Cameron pump. The pressure ran up to something like 1 000 lb. per sq. in., and broke out big chunks of the lining; it was good concrete, too, notwithstanding the authors' suggestion that it was a little weak in spots. We finally got the tunnel pretty tight.

At Shaft 4 on the City Tunnel (which was referred to in the paper, although it has not been discussed by any of the speakers), we drilled into a seam of sand. This sand bed was cut by one of the sump holes and a strong stream of water came up through the hole, carrying with it sand in good-sized grains. We plugged it with a tapered wooden plug, and then, because the stream had amounted to some 200 gal. per min., we decided to grout it. Having made this decision, we kept grouting this seam for about one month, but found that the sand prevented the grout from flowing into any of the holes in any considerable quantity. However, as soon as one hole refused we drilled another and

* New York City.

kept drilling and grouting until the shaft bottom looked like a colander. There must have been several hundred holes in it. Mr. Donaldson.

When we could not get any more grout in we went on sinking and passed through the sand seam without much difficulty. We found that, although the grout had not permeated the sand, it had compacted it so that it resembled soft sandstone and all through the seam we found balls of grout ranging from the size of a man's fist to the size of his head. The leakage into the shaft, which came in through one hole—we did not know how much it would have amounted to if additional holes had been drilled—was cut from 200 to 20 or 30 gal. per min. This shows that grout can be used in sand under certain conditions; but no matter how thin it may be, grout will not permeate sand.

This discussion brings to mind two shafts which the speaker sank in West Virginia before he became connected with the Aqueduct work. Each shaft was about 275 ft. deep and it took two years to sink them. We had to pump about 2 500 gal. per min., and it required nearly 1 000 boiler h. p. all the time. The rock seams were open, and if we had known enough to grout them we could have saved much money and time.

When the shaft reached the seam through which the main flow came in, we found that it carried with it fine coke. Evidently, the particles floated into crevices in the rock in the banks of the river farther up stream, because still farther up the river there were a great many coke ovens, and after every heavy rain coke would be washed in. The biggest seam in the shaft was horizontal; it must have been 4 or 5 in. wide and ran nearly full of water.

One evening, going over to the shaft—something had happened to the pumps—the speaker met a darky coming away. He was carrying a good-sized catfish. The speaker said, "Sam, where did you get that fish?" And the darky replied, "Lord, boss, that came through the big crack in the shaft".

Waldo C. Briggs,* M. Am. Soc. C. E.—The speaker's experience with grouting is confined to a soft ground tunnel which is a part of the Dual Subway System located under Flatbush Avenue, Brooklyn, extending from Prospect Park Plaza to Empire Boulevard. This section is a twin tunnel constructed in a single excavation, about one-half having been driven by means of a shield and the other half by a timber method. Mr. Briggs.

With respect to stopping leaks, three different conditions were presented. Where the shield was started, the material penetrated was a clay carrying a great deal of water. The grouting operations were initiated in this section, and it was the intention to force in only sufficient grout to fill the void caused by the shell of the shield. The concrete behind the shield was placed in rings 31 in. wide by a pneumatic process. The concrete, due to the great quantity of water in the soil, was

* Brooklyn, N. Y.

Mr. Briggs.

porous; this condition, together with the large number of construction joints, produced a very wet tunnel. The result of the attempt to just fill the voids caused by the shell of the shield by means of the grout, so far as stopping leaks is concerned, was a complete failure.

A year or more after this section of tunnel had been completed and the grouting done as described, another attempt was made to stop the leaks by grouting. New holes were drilled through the lining just below the springing line through which grout was forced freely. For a certain distance the tunnel was made dry; but, inasmuch as grout is costly, a limit was set as to the amount of grout used, with the result that where the grouting was so limited, the tunnel is wet. The conclusion to be drawn from this is that porous concrete and construction joints in tunnel lining can be made water-tight by grouting, provided sufficient grout and pressure are used. In the case in question the pressure used was 80 lb. per sq. in.

Another condition, where the shield passed through what was evidently a terminal moraine, the material excavated consisted of boulders and gravel entirely free from any matrix. A section of tunnel at this point, about 100 ft. long, was also wet after the lining had been completed; and, although grout was used freely, the leaks were not entirely stopped, probably due to the fact that the voids in the formation were so extensive it was impossible to force the grout into the concrete.

The third condition was where the timber method of tunneling was followed and where the material penetrated was clean, sharp sand carrying a great quantity of water, the water again causing a somewhat porous concrete and a wet tunnel. Grout forced in through pipes located in the crown of each arch under a pressure of about 65 lb., cut off all water and left a bone-dry tunnel.

The speaker requested the foreman, Mr. Joseph Callahan, who had been in charge of the grouting operations on the work in question, to give him any information which experience had taught would be useful. The following notes were offered in response to this request:

"It is very important to have good hose and valves. The valves used should be cleaned thoroughly every morning or they will become clogged with grout and be useless. The hose and grout kettle should be washed out noon and night. A blow-off valve should be provided at the end of the hose so that if the grout gets blocked in the hose, it can be released. Only cement free from lumps and finely screened sand should be used. Coarse sand and lumpy cement will block the kettle and the hose and valves. When connections are made ready for grouting, try the air to see whether the hole is free. If the air passes slowly, try a thin batch with neat cement at high pressure. The grout pipe in the wall should be cleaned out with a rod. If a hole has been taking grout freely and stops suddenly, it indicates a block; perhaps a stone has fallen into the grout pipe. In such a case, let the grout come back into the kettle,

close the valve at the wall and open the exhaust valve; this should take stones with it. Mr. Briggs.

"All grout pipes should be cleaned at least 75 ft. ahead of the connection in order to release any water which may be behind the wall. As the grout shows ahead of the connection, plug the pipes with wooden plugs. When a hole is not finished at the end of a shift, blow in a little water; this will often keep the hole open until the next shift.

"Before connecting the air pipe to the kettle, blow air through pipe for dirt in the pipe will clog the valves in the kettle and cause trouble.

"Be sure that valves are screwed on tight and that there are no kinks in the hose. If a valve blows off or a hose 'busts', some one is liable to get hurt."

A. J. Sackett,* Assoc. M. Am. Soc. C. E.†—The speaker can add very little to this excellent paper, which covers the ground so thoroughly. Mr. Hogan in the course of his discussion brought out the fact that it is impossible to force grout into poor concrete. The speaker has found that this is true except when the concrete is placed in compressed air, in which case grout washed on the surface of the concrete will be carried through the concrete and fill up the voids thus preventing the escape of air from the tunnel and also decreasing the leakage into the tunnel when the pressure is removed. Experience gained from grouting the Moodna Siphon where 1½-in. pipes were used through the concrete lining shows that much better results can be obtained by using a larger pipe; 2-in. pipes were used on Section No. 63 of the City Tunnel, resulting in a better job of grouting and economy in time. Mr. Sackett.

When electric power is available, it is more economical to develop the compressed air by means of a small portable compressor at the grout tank than to transport the compressed air from a central plant through pipe lines.

In answer to the question as to why hand pumps were used in grouting on the Catskill Aqueduct, it has been the speaker's experience that grouting by hand pumps is a makeshift and only resorted to on small jobs and at places where power is not available or too expensive to obtain.

Concreting the 150-ft. forms on Section No. 63, which Mr. Sanborn has requested the speaker to describe, was a simple process and did not differ in any material way from the methods used to line the tunnel on other sections of the aqueduct. Sand and crushed stone were passed by gravity from bins at the shaft head to a measuring hopper, then through a 12-in. wrought-iron pipe to a 1-cu. yd. concrete mixer mounted at the foot of the shaft. The cement was mixed with water at the top of the shaft, to prevent dust, and dropped though the same pipe with the coarse aggregate. The mixed concrete was transported in 1-cu. yd., side-dump cars, in trains of 8 to 10 cars hauled by electric motors. The forms

* New York City.

† Now M. Am. Soc. C. E.

Mr. Sackett. were built of steel and mounted on trucks which traveled on the same track used by the material cars. The trains of concrete cars were hauled up an incline to the platform of the forms by an electric hoist. The side-wall form traveled in front of the arch form, the two being concreted at the same time. Cars were dumped on the platform, which was about 18 in. above the springing line of the tunnel, and the concrete was shoveled into the forms by hand.

Mr. Cohill. ANDREW A. COHILL,* M. AM. SOC. C. E. (by letter).—In the construction of two tunnels under the East River at 60th Street, known as Route 61, Broadway-Fourth Avenue Subway, there were conditions to contend with that were exceptional, especially the leakage. There was a water pressure exceeding 50 lb. per sq. in. The problem of taking care of the water under Blackwell's Island was a difficult one.

With C. M. Holland, M. Am. Soc. C. E., as Tunnel Engineer, and M. H. Freeman, M. Am. Soc. C. E., as Resident Engineer in direct supervision of the work, precisely the same methods as those described in this paper were used, and the results obtained were excellent. Approximately, 11 000 ft. of tunnel was grouted, of which 5 000 ft. has a cast-iron lining. The leakage before grouting was 800 to 900 gal. per min.; this was reduced, by grouting, to approximately 25 gal. per min.

The writer wishes to compliment the authors on the excellence of this interesting and valuable paper.

Mr. Mayell. A. J. MAYELL,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer was much interested in the grouting methods used on the water tunnels, as described by the authors, and especially in the use of grout of varying consistencies, and can testify that the methods are efficacious in stopping leaks not only in water tunnels but in the New York subways.

Similar methods were recently used with great success by M. H. Freeman, M. Am. Soc. C. E., at the Broadway Station of the Canal Street Crosstown branch of the Broadway-Fourth Avenue Subway where considerable leakage developed after construction was completed, and ground-water which had been lowered by pumping during construction regained its original elevation.

This station is entirely below mean high water, and was therefore enveloped in water-proofing consisting of two layers of brick laid in asphalt mastic. Where it passed under the Interborough Subway in Lafayette Street and the Centre Street Loop in Centre Street, the water-proofing, instead of being carried over the roof of the new structure, was lapped on to the water-proofing on the under side of the overhead structures. It is probable that a slight subsidence of the mastic together with pressure from the water head caused this water-proofing lap to open slightly and admit ground-water into the subway.

* New York City.

Before the ground-water had risen to its normal level, the station finish was placed and the subway put in operation, so that when the leakage developed the water was carried by the hollow tile and through air spaces a considerable distance from the points of entry and flowed across the platforms causing annoyance and inconvenience to passengers. Mr. Mayell.

It was decided to attempt to stop the leaks by grouting from the inside of the structure; holes were drilled through the interior walls in the general direction of the water-proofing lap, and grout was injected under pressures of from 30 to 70 lb. per sq. in. The total leakage at the start was about 160 gal. per min., most of it coming in under the two overhead structures. The grouting reduced the leakage to less than 1 gal. per min.

The grouting was done under difficult conditions. The station platforms and passageways from which grouting operations had to be carried on were crowded during rush hours and great care had to be taken to prevent accidents. Moreover, special precautions were necessary and constant vigilance had to be exercised to prevent the grout from filling the hollow tile, air spaces, lighting conduits, and drains. Pressures had to be kept fairly low to prevent damage to the structure and to the overhead structures.

The great force that may be exerted by grout under pressure is indicated by the fact that the Interborough Subway, which during the construction of the structure below it had settled from 1 to 3 in., was raised from 1 to 1½ in. by the grouting. This occurred after all the holes under the Interborough Subway had been grouted and the process of "squeezing" to complete the seal was in operation. The squeezing was done by a hand grout pump with a pressure gauge attached. The pressure generally used was between 50 and 60 lb. per sq. in., but on this occasion the grout was going in very slowly and the pressure was increased, as it was thought that the space between the two structures had been quite thoroughly grouted and that the grout could not exert pressure on any great area of the overhead subway bottom.

No damage was done to the overhead structure; on the contrary, the lifting eliminated the greater part of the depression in the inverts of the Interborough structure which the operating company had thought might affect the drainage of its tracks.

That the results obtained were so successful was due to the intimate knowledge which Mr. Freeman had of the action and effect of grout of different consistencies and his constant attention to this detail.

MILTON H. FREEMAN,* M. AM. SOC. C. E.—The authors of the paper deserve credit for presenting data of great value. Such data are most difficult to obtain as they involve many trials and experiments on work Mr. Freeman.

* Valhalla, N. Y.

Mr. Freeman.

of considerable magnitude. The following detail considerations are submitted as being of possible interest.

Placing and Protecting Grout Pipes.—Reference has been made to the necessity of a good concrete lining in order to make grouting a success in securing a water-tight tunnel, and it may be of interest to consider some particular features relative to the placing and protection of grout pipes as an aid in securing good concrete. The placing of pans and pipes subsequently to be grouted requires the utmost care and forethought; they should be placed with some method in mind for finally filling them with grout, not simply with the idea of taking care of incoming water. A careful record should be made showing what particular function each pipe serves at the rock end so that, later, grouting can be done intelligently. Pipes are sometimes disturbed by the placing of a heavy mass of concrete, and a most careful inspection of these pipes is necessary during concreting. All drain pipes, of necessity, have to be brought through the forms, and it is far better if the vent pipes are placed in this way so that they may be cleared of any concrete that may become lodged in them while the concrete is still soft. Pipes draining large leaks usually free themselves of any mortar that may work in from the concrete, but pipes set to drain small drips are easily clogged, and it is not an uncommon experience to find one that was placed to pick up a small drip perfectly dry, while the leak finds its way through the body of the concrete at some near-by point. A material aid in such cases is the washing out of the bottom of the drain pipe by inserting a small ($\frac{1}{2}$-in.) pipe connected to the water line, or air line if water is not available, after the manner of blowing out drill holes before loading in a rock heading. This must be done with extreme care after the concrete is from 1 to 2 ft. above the rock end of the pipe. This procedure has often saved pipes that started to clog at the rock end.

Shaft Ring Pans.—The ring pans in shafts, as described by the authors, have a double service: First, they furnish channels for grouting any shrinkage cracks between the concrete and the rock and possibly cracks in the rock itself; second, and probably their most positive function is to collect seepages that are not easily discernible, and consequently are not taken care of with individual pans on the wet sides of a dripping shaft, and which follow up behind the concrete until collected in the ring pan. Where these pans have been used, it is a most common occurrence to find the down drain pipes dripping after all the concrete has been placed. Since leaving the work on the Aqueduct the speaker has found in certain instances, where shaft lining has been placed without the use of these pans, that there is a greater tendency to leakage at horizontal joints. The pans are preventatives against such leaks.

It has been thought by some that it might be worth while first to grout these rings with thin grout under high pressure, to penetrate the

rock seams, and afterward blow them out and close them with thick grout. Mr. Freeman.

Displacing Water with Grout.—Recently, the speaker has had occasion to do some grouting where water had to be displaced by grout, and found that when the volume of cement to be injected is comparatively small and the necessary grouting pressures not excessive (60 to 80 lb.) the hand pump has proved to be of excellent service, as it is under instant control, wholly eliminates the ill effects of the after-blast of air, and displaces the water by grout without disturbing and mixing the two. From 100 to 150 bags of cement can be injected with the hand pump per 8-hour shift, under favorable conditions. Grouting to displace water with an air-discharging grout tank is most difficult to accomplish without the formation of considerable laitance. Shutting off the tank pressure when the hose kicks or when the tank gauge pressure drops is of value, but when this phenomenon takes place, a charge of air has already gone through the hose, to stir up the grout with the overlying water. Keeping the grouting pressure as low as possible is a worth-while precaution, even at the expense of clogging the hose once in a while.

Volume of Set Grout in Place.—The question of the ratio between the volume of mixed grout and the volume of set grout in place is often required, sometimes as a payment quantity. The contracts of the Board of Water Supply very wisely avoided this, as it has been found that there is no fixed, definite ratio. Not only does the volume vary with the quantity of water and the quantity and kind of sand used, but the degree to which the grout mass is shaken in the process of placing. Experiments on small quantities indicate that, due to the latter cause alone, a variation of 20% may be found. If necessary to determine this ratio, experiments involving the particular kind and quantity of sand used and the quantity of water utilized for mixing will probably give the most reliable data.

Grouting Overhead Closures.—One feature of grouting which may be of interest is the method of making overhead closures where the surplus water from the grout is trapped, that is, cannot be disposed of by increasing the grouting pressure. Concrete structures are often built up under other structures, where it is essential to secure a good bearing, possibly a joint that will be water-tight. A concrete arch placed inside a steel or iron lining is another illustration.

Certain features in regard to this kind of grouting have been worked out as follows:

1.—The thicker the grout the less surplus water will be thrown off.
2.—Practically all this surplus water is thrown off and pools form on the top of the grout within one hour after placing.

Mr. Freeman.

3.—If sand is used, the presence of a small quantity of loam (as little as 5%) increases noticeably the amount of laitance at the top.

4.—A second, third or possibly a fourth grouting may be necessary finally to fill the space occupied by the surplus water from the grout ejected previously.

5.—Vent pipes should be set to take advantage of any small summits; if the overhead structure is of concrete, summits may be chipped out. Arch or roof concrete forms often sag when concrete is placed on them, but the vent pipes should not be allowed to sag with the forms.

If all pipes are cleaned out as soon as the grout has partly set, the same pipes can be used for subsequent groutings. This necessitates individual vent pipes to each summit. A second grouting usually leaves only a very small space unfilled, but sometimes grout can be injected at a third or fourth trial. Neat cement grout is commonly used after the first grouting. Higher pressures can be used with advantage as the space fills up, if the structures themselves are strong enough to withstand such pressures. The foregoing, of course, applies only to places where it is essential to obtain a complete contact between the two structures.

Grouting Undisturbed Sand.—Several attempts to grout undisturbed sand have come under the speaker's notice, but all such attempts were ineffectual, even though thin grout and high pressures were used. A little cement can sometimes be injected into gravel that contains no fine sand as a filling material.

Mr. Riker.

C. L. Riker, Jr.,* Assoc. M. Am. Soc. C. E.—High-pressure grouting operations were resorted to on the lining of about 1 200 ft. of the Catskill Aqueduct in the section south of Twenty-fifth Street, New York City, following the development of outward leaks through numerous small cracks which opened in the concrete lining during the initial hydraulic test of this section. These cracks were due to the compression or shifting of the soft seamy rock through which this section of the tunnel passed.

This section had previously been grouted by the use of both low and high pressures, but numerous hair cracks in the lining indicated that the original high-pressure grouting had not penetrated the minute seams of the rocks a sufficient distance or thoroughly sealed and formed an unyielding backing to the lining. Ground-water flowed freely from many of these cracks. It was necessary, of course, to stop this flow entirely prior to commencing grouting operations. These leaks were

* Brooklyn, N. Y.

effectively stopped by chipping out a groove along the line of the crack and caulking the groove with lead wire. Mr. Riker.

Threaded pipes, 1½ in. in diameter, were rolled tightly with cotton drill and driven into 2¼-in. holes bored in the concrete lining. This formed an entirely satisfactory and tight joint between the pipe and the tunnel lining.

In regrouting this section, 9 500 ft. of 2¼-in. hole, or a little less than 8 ft. of hole per foot of tunnel, was driven, and 2 400 connections were made to these holes, or 2 connections per foot of tunnel treated. More than one connection was made to many of the pipes, and some were only used for venting purposes. About 44 000 cu. ft. of very thin liquid grout was forced through these connections, using air pressure at from 250 to 350 lb. per sq. in. at the displacement tank.

Generally speaking, apparently the grout was effectively prevented from re-entering the tunnel. However, a peculiar substance incident to the grouting accumulated within the tunnel. A heavy grayish custard-like mass was formed, which, when quiescent, became almost as hard as cheese; when agitated it became soft and liquid, and was carried in solution by a sluggish flow of water in the invert. A settling basin was formed on a slope down stream from the grouting operations, and about 800 cu. yd. of this soft waste matter accumulated as a result of 45 000 cu. ft. of liquid grouting.

A considerable amount of solidified droppings accumulated adjacent to the grout tanks, caused by spillage, venting, and blowing out the grout hose. Apparently, most of the grout so wasted solidified adjacent to the grout machine even though dropped into the slowly flowing water in the invert as well as on the lower quarters of the tunnel.

It is hard to account for the enormous quantity of this custard-like material, and it would be interesting to know whether the grouting process generally results in similar waste products, and, if so, of what this waste material consists.

Benjamin A. Howes,* Assoc. M. Am. Soc. C. E.†—An important point as to the fine grinding of cement is involved. Theoretically, as the authors intimate, it would seem that very finely ground cement would percolate more readily into the finer cracks and crevices as grout. Likewise, it would seem that if the cement were ground extremely fine, it could be carried into the interstices of sand and gravel as grout to consolidate them in place. The authors' statement indicates that experience shows this to be impossible and that standard fineness has been found to be more practical. Mr. Howes.

The mechanical features of setting and hardening explain this seeming clash between theory and practice. The finest particles are hydrated by the mixing water almost immediately, because the water reaches the entire mass at once. Particles of appreciable size are reached by the

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† Now M. Am. Soc. C. E.

Mr. Howes.

water only on the exterior surface, and hydration proceeds by the attack of successive layers and to some extent by percolation of water into crevices of the particles. Ordinarily, the still coarser particles are hydrated on their exterior surfaces only, and these coarser particles with some 10 or 15% of inert constituents often comprise 40% of the cement. They act as reserves or factors of safety and will hydrate in time if needed to consolidate the mass.

Therefore, for grouting it is essential that a large part of the cement be ground so coarsely that it will not hydrate until it has reached its final resting place and will then by slow hydration consolidate the fluid that has been the carrier.

When a porous tunnel lining is under air pressure, grout will carry the coarse particles into the pores to some extent and there they will hydrate and stop the flow of air or water, but this only in very small degree fills and consolidates the total porosity. There is one place where these reserves of coarse particles demonstrate their office very plainly—in the case of small cracks in a concrete wall retaining water. Sometimes the cement reserves seal the crack in place and sometimes percolating water brings dissolved cement to the surface, where evaporation condenses it to seal the crack.

The speaker has found these unhydrated coarse particles of cement in hydraulic structures years after construction. In one case a tunnel lining, in another, a dam which had been in use 4 or 5 years showed unhydrated particles of cement from $\frac{1}{1000}$ in. to $\frac{1}{2000}$ in. in diameter. These particles had the characteristic cellular structure of Portland cement clinker. Hydration had taken place around the surface of the particles, and the interiors of such particles appeared to be ready to hydrate if water was introduced through a crack or otherwise.

Mr. Belzner.

THEODORE BELZNER,* ASSOC. AM. SOC. C. E. (by letter).—Milton H. Freeman, M. Am. Soc. C. E., states that "several attempts to grout undisturbed sand have come under the speaker's notice, but all such attempts were ineffectual."

It might be of interest to state that the writer recalls a few experiments made some years ago, during the earlier construction of the subway in the lower section of New York City, by forcing grout into undisturbed sand under pressure. When the material into which the grout had been injected was excavated, it was found that the cement had not been distributed uniformly. The material was lumpy and of no value whatever.

Mr. Zipser.

M. E. ZIPSER,† ASSOC. M. AM. SOC. C. E. (by letter).—It is very gratifying to the authors that the paper has been discussed by those so well qualified to give expert opinion on the subject.

* Brooklyn, N. Y.

† New York City.

Mr. Zipser.

Mr. Ridgway's point that a good concrete tunnel lining is essential in order to accomplish successful results in subsequent grouting, is very important. This matter is also given emphasis by Mr. Briggs' discussion of the case where costly grouting had to be resorted to in order to stop the leaks in a tunnel lining the concrete in which was porous, due, apparently, to lack of proper provision to keep the water away from the concrete until the latter had set.

Mr. Smith's remarks are very much appreciated by the authors. All who had the good fortune to be associated with Mr. Smith in the building of the Catskill Aqueduct know that the guiding genius and inspiring personality of the Chief Engineer stimulated every one to put forth his best effort.

Mr. White very properly calls attention to the fact that it was not the grouting alone that made the pressure tunnels water-tight, but that the concrete lining itself was, to a large extent, responsible for the tightness. Just to what degree the grouting and the lining each contributed to the results obtained, is difficult to state. When the plans were being prepared, it was thought that it would be possible to plug all the cracks and crevices in the rock and impregnate any porous rock surrounding the tunnel with grout, and that the concrete lining after having served the purpose of a bulkhead for the grouting would provide a smooth coating to produce a high coefficient of flow. In actual practice, however, it was found to be impracticable and of no advantage to attempt to force grout into all the fine seams in the rock. The results aimed at could be obtained by the simple but effectual methods developed during the progress of the work.

Mr. Hogan questions a statement of the authors, namely, that one of the purposes of the low-pressure grouting in the pressure tunnels was "to fill any porous or honey-combed spots in the concrete." What the authors had in mind in this connection was the filling with grout of any porous concrete in the top of the arch, where the consistency of the concrete when placed was relatively dry, and there was no opportunity for working it. For this purpose the grout was very effective in making the arch concrete dense and solid.

Mr. Freeman, in his discussion, brings out several important points in connection with the details of grouting. Mr. Freeman's wide experience and signal success in grouting work under varying conditions makes his contribution an especially valuable one.

In conclusion, the authors wish to thank all who have discussed the paper. Their comments and suggestions, and their accounts of experience on other work, will be of great value to any one who may be called on to engage in grouting operations.

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Paper No. 1448

A NOVEL METHOD OF REPAIRING A SWING BRIDGE*

By Herbert C. Keith, M. Am. Soc. C. E.

With Discussion by Messrs. T. Kennard Thomson, Guy Pinner, Shortridge Hardesty, and James B. French.

Synopsis.

In the course of the regular inspection of the bridges of The Connecticut Company in 1912, the bottom chords of the swing span of the Washington Bridge over the Housatonic River were found to be in a dangerous condition because of the great loss of section by corrosion, due to the street drainage having been discharged upon the chords for years.

This bridge carries a highway with a single electric railway track, and being on the direct route from New York to Boston, the traffic is moderately heavy. There is no other bridge within seven miles to which this traffic could be diverted, and as the Housatonic at this point is a navigable stream, any material interruption either to navigation or highway and railway travel during reconstruction was prohibited.

This paper describes the methods used to renew the chords, with a very material saving of time and money, without impeding traffic. A system of temporary adjustable timber substitute members was devised to carry the bottom chord stresses while the permanent chords were being renewed, allowing the free use of the bridge for half the width

* Presented at the meeting of April 7th, 1920.

of the roadway at all times, and also permitting the span to be opened as required for river traffic.

Location.

Washington Bridge, connecting the Towns of Stratford and Milford, Conn., crosses the Housatonic River about 1½ miles above its mouth, and carries the main highway between Bridgeport and New Haven. At this point, the electric railway of The Connecticut Company has a single track in the highway, though the line is double-tracked for most of its length.

Original Construction.

The bridge was built in 1893 by Dean and Westbrook, as a highway bridge, for Fairfield and New Haven Counties, the steel having been fabricated by the Berlin Iron Bridge Company. This structure consists of a 203-ft. swing span with a 232-ft. fixed span at each end. The fixed spans have through pin-connected triangular trusses with curved top chords, each span with eight 29-ft. panels subdivided into sixteen floor panels of 14 ft. 6 in.

The swing span has through pin-connected trusses, Fig. 1, with an 18-ft. rectangular pivot panel flanked at each end by three triangular panels of 30 ft. 10 in., subdivided into six floor panels 15 ft. 5 in. long. The top chord of each arm of this span is curved, with a height varying from 20 to 33 ft. This swing span is entirely center-bearing, but has a turn-table with six wheels running on a circular track to provide for unbalanced load. The original bottom chords were 7-in. channels, latticed, 13¾ in. back to back, with splices near the main panel points. In the pivot panel were two additional channels between the outer ones, the latter being continuous with those of the remainder of the chord.

In all the spans the trusses are 19 ft. 6 in. on centers, the highway being carried between the trusses by 24-in. plate-girder floor-beams hung below the chords by loop hangers passing around the pins and down to tie-plates under the bottom flanges. The plank floor is supported by I-beam and channel stringers resting on cast-iron chairs on top of the floor-beams.

One of the designers of this bridge said, many years later (after having had extensive experience in bridge construction), that he doubted if there was a pound of surplus material in the original bridge in excess of that required to meet the specifications under which it was designed.

Earlier Repairs and Alterations.

Originally there was no track on the bridge, but one was laid later, its center line in 1913 being 3 ft. 11 in. south of the center line between trusses, as seen in Fig. 2.

The bridge has been strengthened or extensively repaired or altered several times, one of the more recent changes being the addition of a 4-ft. sidewalk on brackets outside of the north trusses. At the same time, The Connecticut Company renewed its track stringers, increasing their capacity, and cover-plates were added on both flanges of the floor-beams; additional floor-beam hangers were put on, and some repairs were made to the bottom chords of the main trusses. This work was required not only to provide for increased loads, but because of serious corrosion found on many parts of the bridge.

Condition in 1912.

Examination of this structure in the course of a general inspection of the bridges of The Connecticut Company in the fall of 1912 revealed the fact that serious corrosion had taken place in the bottom chords of the swing span. Although the bottom chords of the fixed span (being of eye-bars) had suffered no material loss, the discharge of drainage from the roadway through slots in the under side of the wheel guards had been very destructive to the latticed channel chords of the trusses of the swing span. The chords of both trusses of this span were in such condition for nearly the whole length that their immediate renewal was necessary, or else the use of the bridge would have had to be discontinued. As indicated by Fig. 3, in several places, the top or bottom flange of a channel was nearly or entirely gone, and at other places large holes were rusted through the web. Corrosion was general for the whole length of both trusses, but was greatest near the pivot pier, and the inner channels had suffered more than the outer ones.

Some of the pin-holes had been enlarged by rust or badly distorted by overstrain (see Fig. 4). Unfortunately, no satisfactory photograph was obtained of the worst case of such enlargement, where rust had enlarged the pin-hole more than 1 in. beyond its normal lines.

These facts made it evident that the unit strains on the reduced chord sections, due to the dead load when the bridge was swinging, far exceeded any possible live load unit strains, and the chief danger of failure was by "breaking its back" when swinging rather than by weakness for carrying traffic.

FIG. 1.—GENERAL VIEW OF WASHINGTON BRIDGE OVER HOUSATONIC RIVER.

FIG. 2.—EAST PORTAL OF WASHINGTON BRIDGE OVER HOUSATONIC RIVER.

Factors Affecting the Method of Repair.

The first method of procedure to suggest itself was to construct a temporary bridge with a jack-knife draw on one side of the present bridge, and to divert the traffic thereto. Then the draw of Washington Bridge could be swung open and be blocked up on the fender pier to allow the renewal of the chords. The great expense involved in this method quickly forced the consideration of other possibilities—and quasi-possibilities.

There is no bridge or ferry across the Housatonic River below the Washington Bridge, but the main line of the New York, New Haven and Hartford Railroad has a four-track bridge over the river about ¼ mile up stream. The location and construction of this bridge and its approaches, however, are not such as to allow easy or economical diversion of highway or trolley traffic to it. The nearest other public means of conveyance across the river is 7 miles distant; to divert traffic to that route would have added about 12 miles to the 17 miles by direct route from Bridgeport to New Haven.

Both passenger and merchandise traffic across the Washington Bridge is moderately heavy on the electric railway and also on the highway, as the bridge is on the most direct route between Bridgeport and New Haven and between New York and Boston. The inconvenience and expense that would have been caused by the interruption of traffic would have been great. Some of the traffic would have been discontinued or sent by railroad, but most of it would have been diverted to the crossing 7 miles up the river, with an average increased haul of about 12 miles.

The river traffic is lighter, but the Federal Government gives such attention to the maintenance of its water traffic that it is more practicable to stop entirely thousands of travelers on a highway or railway over a navigable stream rather than prevent for a single hour the passage of a yacht, or a tug with a scow, across the line of such highway or railway. It was learned, however, that Nature does each winter what the Government prohibits man from doing—closes the river to navigation for about three months. Records kept for several years show that the ice became so thick as to prevent navigation from December or January to March or April of each winter for which records were available. It was thought that the War Department might be persuaded to allow piles to be driven through the ice, provided that such a

man-built barrier were not allowed to remain to interfere with theoretical open navigation longer than the ice presented its prohibition to actual navigation. However, uncertainty of obtaining such permission, as well as the cost of this method of making the repairs, emphasized the desirability of further study of other methods.

It may be observed from Fig. 5 that though the swing span has six floor-panels at each end, yet, structurally, it has only three panels in each arm, each truss panel having its bottom chord divided into two parts corresponding in length with the floor-panels and, of course, having equal stress. As only about one-third of the deflection of the draw when swinging was taken up by the end lifts, nearly all the dead load was carried by the trusses to the center bearing even with the bridge closed, very little of this dead load being supported at the ends. This resulted in a constant compressive strain in the whole length of the bottom chords except the end panels, whether the bridge was swinging free or was closed and carrying the maximum live load. Only a small tensile stress was possible even in the end panels. The plans were examined, and careful inspection was made of the structure itself to ascertain whether careless erection or the numerous repairs and strengthenings might have changed conditions from those shown on the plans. It was found that there were small spaces on each of the main pins where jaws of compression members could obtain bearings.

Consideration of these facts led to the adoption of a plan for carrying the bottom chord strains by temporary substitutes while the chords were being renewed, and study of the problem brought conviction that this was not only feasible but a surprisingly economical solution. It avoided the cost of falsework and of constructing a temporary crossing for a detour.

Stress Requirements and Assumptions.

It was assumed that raising the ends of the swing span relieved each truss of that portion of the dead load which was concentrated at the end panel points, L_0, leaving the remaining dead load carried as if the bridge were swinging free. The live load was considered as carried by a continuous truss on three supports. With these assumptions, if (*a*), the railway traffic were stopped during the progress of the work; (*b*), the sidewalk were removed; and (*c*), the highway traffic were confined at all times to the half-width of the bridge away from the truss on which the work was being done, the maximum stresses in the bottom

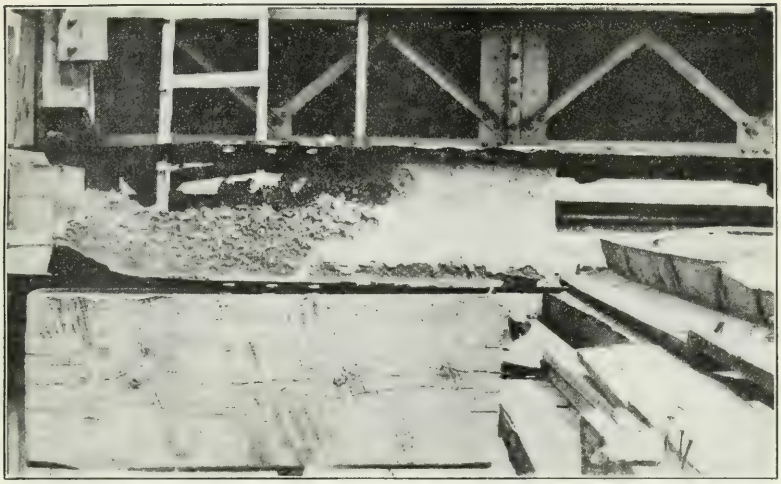

FIG. 3.—INNER CHANNEL OF BOTTOM CHORD OF PIVOT PANEL, SOUTH TRUSS.

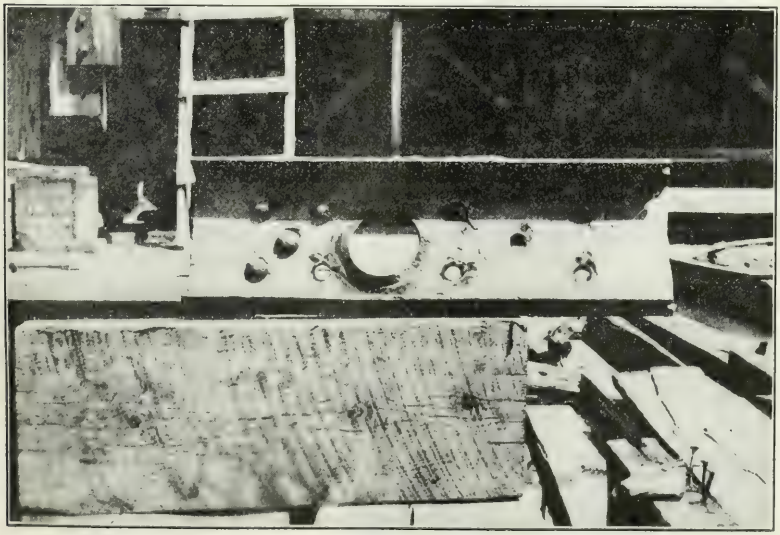

FIG. 4.—INNER CHANNEL OF BOTTOM CHORD OF SOUTH TRUSS AT L_0.

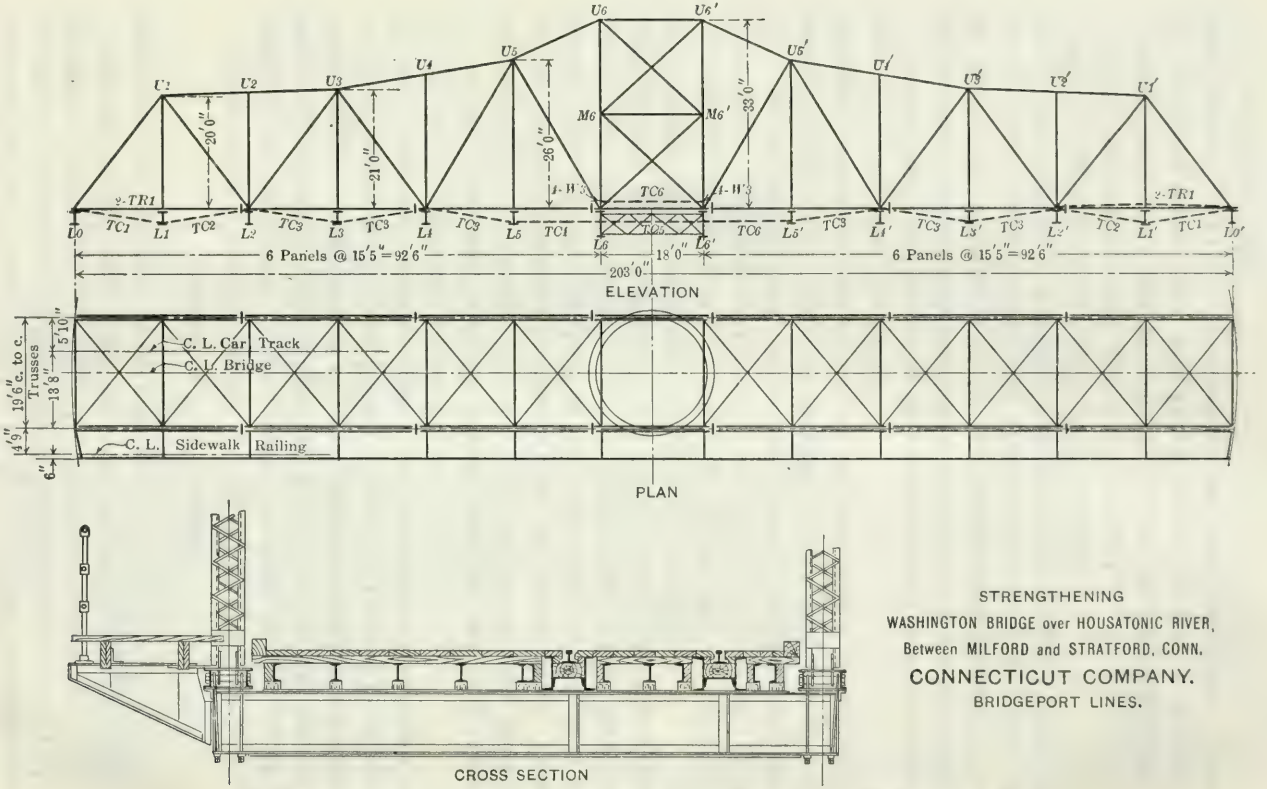

FIG. 5.—ELEVATION, PLAN AND CROSS-SECTION, SWING SPAN, WASHINGTON BRIDGE.

chords of the north truss would be: In L_0-L_1-L_2, 6 600 lb. compression or 4 300 lb. tension; in L_2-L_3-L_4, 49 000 lb. compression; in L_4-L_5-L_6, 105 000 lb. compression, and in L_6-L_6', 118 000 lb. compression.

Stresses in the south truss would be slightly less, because there were no sidewalk brackets on the south side.

Outline of Plan Adopted.

Based on the foregoing stresses, a plan was prepared for entirely relieving the steel chord of all compressive strain by the use of a temporary adjustable timber substitute chord (see Fig. 5), each full panel (except as modified near the pivot panel) having inclined struts with jaws bearing against the pins at the main panel points, L_0, L_2, and L_4, their other ends abutting against the intermediate floor-beams at L_1, L_3, and L_5, near their bottom flanges.

In the pivot panel, a double timber strut was required, placed horizontally with one timber at each side of the latticed strut, because of the latticed longitudinal strut between the cross-girders. These timbers were placed at the level of the lower ends of the timbers in other panels, and the timber in the half-panel at each side of the pivot panel was also placed horizontally and abutted against the cross-girder at L_6 directly opposite the double strut of the pivot panel.

As the temporary chord did not bear against the pins, L_6, next to the pivot panel, an unbalanced thrust was left for the batter posts, U_5-L_6, in the adjacent panel at each end, which must be transferred across the pivot panel. This was provided for by a timber strut (also adjustable) above the level of the pins, and bearing against the vertical posts, U_6-L_6, over the cross-girders.

As the chord stress is equal in both parts of a full panel and as the two temporary struts which were used to take the strain abutted against the pins at the main panel points and against opposite sides of the floor-beams at the same height at intermediate points, there was no strain in the floor-beams due to the temporary chord (or substitute), but merely a slight additional tension in the hangers.

In the full panel at each end of the bridge, L_0-L_1-L_2, tension was provided for by two adjustable rods passing over the intermediate pin, L_1, and connected by yokes with U-rods around the pins at the main panel points, L_0 and L_2. No provision was made for tension in any other part of the chord.

PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
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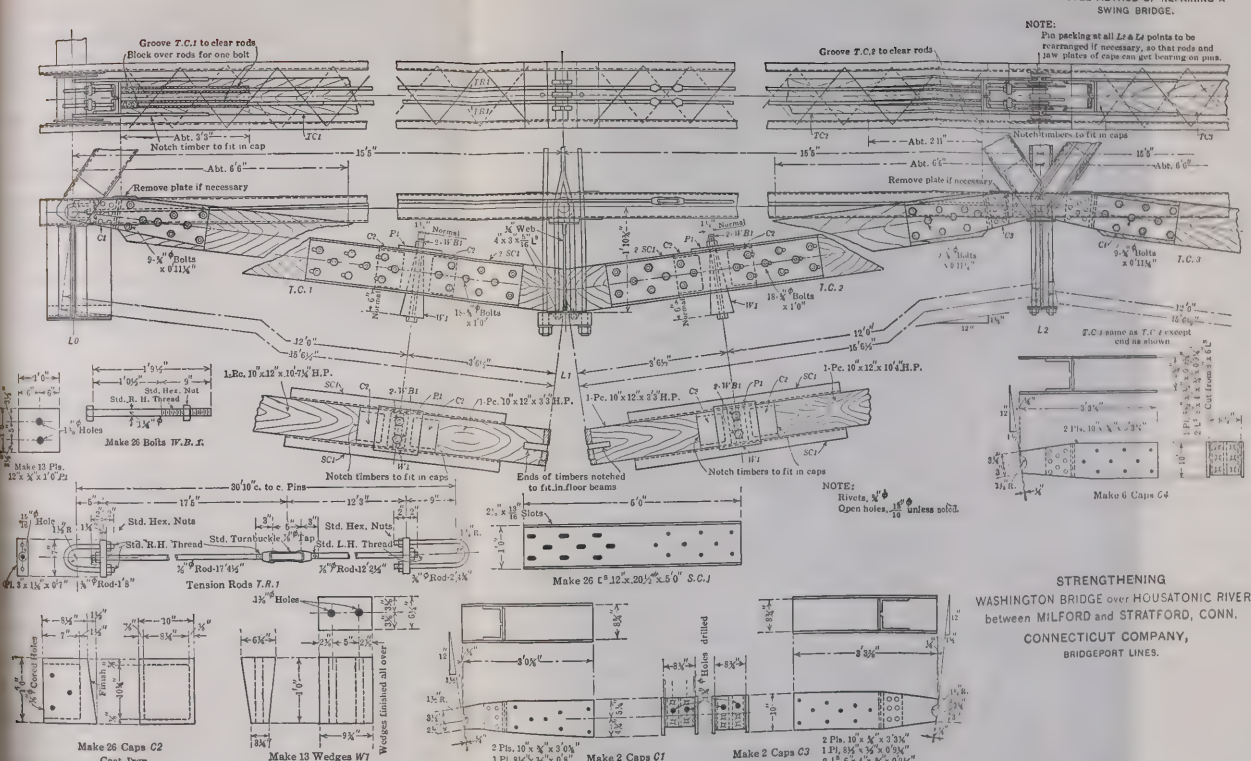


STRENGTHENING
WASHINGTON BRIDGE over HOUSATONIC RIVER
between MILFORD and STRATFORD, CONN.
CONNECTICUT COMPANY,
BRIDGEPORT LINES.

All the weight of the bridge when swinging, and (by the assumption made) nearly all of the dead load and a large part of the live load when the draw is closed, are delivered to the cross-girders at L_6 by the bearing of the bottom chord channels upon the cross-girders, whence it is carried to the center bearing, except for a small amount of unbalanced load carried by the turn-table wheels. It is evident that if the chord channels were removed entirely from the top of the cross-girders and no other provision was made for supporting the bridge, it would drop until a bearing of the vertical or batter posts was reached on the cross-girders. To prevent such a drop, four wedges $\frac{5}{8}$ in. thick were driven under each pin, L_6, just inside the webs of the channels forming the vertical post, and outside of those forming the main diagonal, U_5-L_6.

The longitudinal latticed struts between the cross-girders under the trusses were corroded very badly and had to be renewed; and readjustment was required in some of the truss members.

Details of Design.

In the first five sub-panels from each end of the span (see Plate XV), each compression member was composed of two pieces of 10 by 12-in. hard pine timber, separated by a forged steel wedge sliding vertically between cast-iron caps on the timbers, thus allowing for adjustment in the length of the member.

The lower end of this strut was trimmed so that it would bear solidly against the web and bottom flange of the intermediate floor-beam, while the upper end was cut down to a width of $8\frac{3}{4}$ in., and sawed off square to butt against the diaphragm of a steel cap with $\frac{3}{4}$-in. jaw-plates bolted one on each side of the timber. These jaws projected beyond the end of the timber and diaphragm, and had their ends cut out semi-circularly to fit against the main truss pin. Between these jaw-plates, and bearing against the end of the timber, a $\frac{1}{2}$-in. diaphragm plate was attached to the jaws by two angles, 8 (or 6) by 4 by $\frac{3}{4}$ in., the rivets being countersunk in the diaphragm so as to present a flat surface for bearing on the end of the timber.

The wedge was $3\frac{1}{2}$ ft. from the floor-beam end of the strut, and had two planed surfaces, each with a slope of 1 in 8, sliding between planed surfaces of the cast-iron caps on the timbers. Two $1\frac{1}{4}$-in. bolts, passing through the wedge and through a washer plate 12 in. square

by $\frac{3}{4}$ in. thick, were used to draw the wedge into place and to prevent it from slipping. Two scabs of 12-in. channels, 5 ft. long, were bolted to the strut, one on each side, to maintain the strut in proper line. These scabs were secured to the component timbers by nine $\frac{3}{4}$-in. bolts for each timber, three of which passed also through the cast-iron cap. The holes in the scab channels for the bolts for the lower, and shorter, timber were $1\frac{3}{16}$ in. in diameter, while those for the bolts for the other timber were slotted to $2\frac{3}{8}$ in. in length. These bolts were left loose until the wedges were drawn up the full amount, and then tightened. The slopes of 1 in 8 (or $1\frac{1}{2}$ in. per ft.) were adopted for the wedge as nearly balancing the friction, if the faces of the wedge were well greased, the coefficient of friction being estimated at about 0.15.

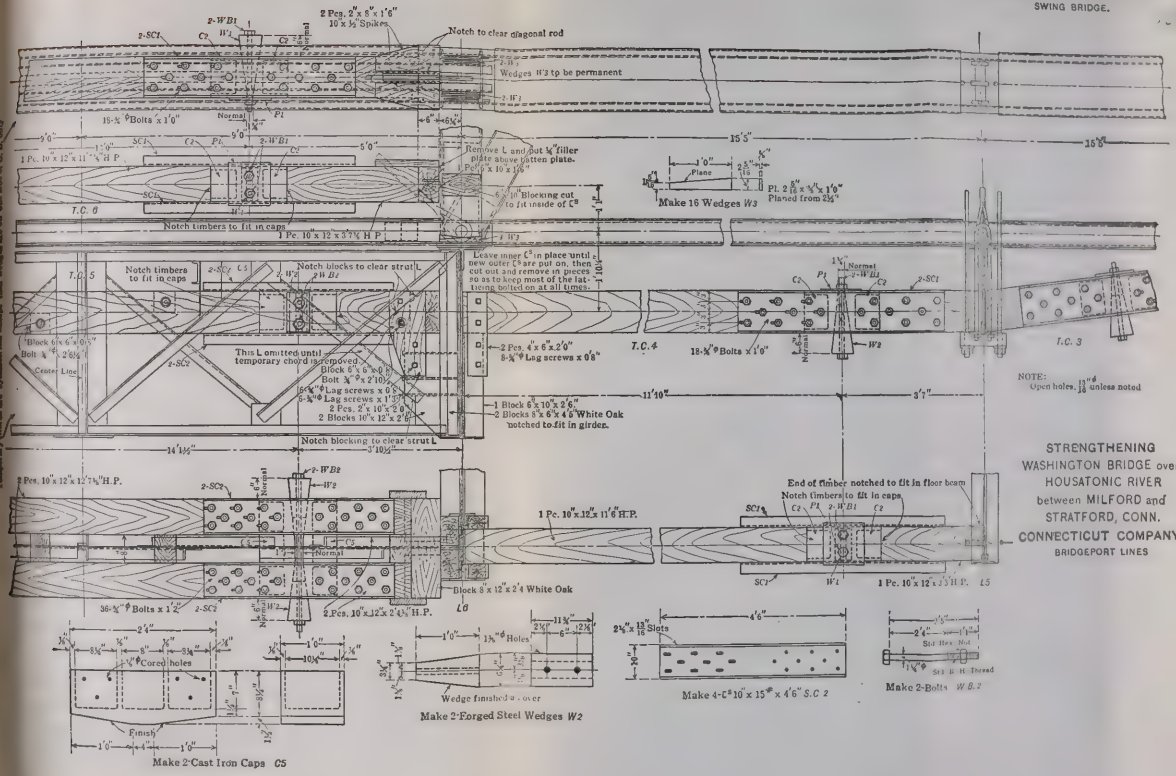
Later experience confirmed this estimate, for when the bolts were slacked off, it required the tap of a hammer (in some cases a hard blow) to loosen the wedges.

In the half-panel next to the pivot panel (Plate XVI), the strut was made similar to those in the first five half-panels except that instead of the jaw connection for the pin at L_6, the strut was placed horizontally with a square end of the timber abutting against the web of the cross-girder under the pin, L_6. Blocking underneath and at both sides of the strut held it securely in place.

In the pivot panel a double strut was used. It was laid horizontally and level with those in the adjacent half-panels, but with one line of timbers on each side of the permanent latticed steel strut between the cross-girders, which (except one diagonal temporarily omitted) had been previously renewed. These timbers were bolted together through spacing blocks at three points, and had double cast-iron caps for wedges sliding horizontally, the two wedges being drawn in by two $1\frac{1}{4}$-in. rods passing through both wedges. At the expansion joints, 10-in. scab channels, similar to the 12-in. channels in the other panels, were used, but they were placed on top of and underneath the timbers. Blocking held the struts in place at points of bearing against the cross-girders and to distribute the pressure of the two struts of this panel so as to oppose the single struts of the adjacent half-panels without straining the webs of the cross-girders.

A strut was placed 13 in. (on centers) above the chord in the pivot panel to take the unbalanced thrust of the diagonal posts of the adjacent panels. Its construction was similar to that of the other

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII. No. 1448.
KEITH ON
A NOVEL METHOD OF REPAIRING A
SWING BRIDGE.


Temporary struts are symmetrical except that wedges are on one side of C. L. only
2 Peg, 2" x 8" x 1'6" 10" x ½" Spikes
Notch to clear diagonal rod
2-WB1
2-SC1
C2
W1
P1
Wedges W3 to be permanent
2-W3
15-¾" ϕ Bolts x 1'0"
Normal
1 Pc. 10" x 12" x 11¾" H.P.
5'0"
15'3"
15'8"
Remove L and cut ¼" filler plate above batten plate.
1 Pc. ¼" x 10" x 3'6"
5'-10" Blocking cut to fit inside of L
Make 16 Wedges W3
Plane
Pl. 2 5/16" x ¾" x 1'0" Planed from 2½"
T.C. 6
Notch timbers to fit in caps
1 Pc. 10" x 12" x 3'7¾" H.P.
T.C. 5
Notch timbers to fit in caps
Notch blocks to clear strut L
2-WB1
Leave inner L in place until new outer L is put on, then cut out and remove in pieces so as to keep most of the latticing bolted on at all times.
T.C. 4
T.C. 3
1¼" Normal
2-SC1
18-¾" ϕ Bolts x 1'0"
W2
2 Pcs. 4" x 6" x 2'0"
8-¾" ϕ Lag screws x 0'8"
Block 6" x 6" x 0'5" Bolt ¾" x 2'6½" Center Line
This L omitted until temporary chord is removed
Block 6" x 6" x 0'8"
Bolt ¾" x 2'10½"
6-¾" ϕ Lag screws x 0'6"
6-¾" ϕ Lag screws x 1'3"
2 Pcs. 2" x 10" x 2'0"
2 Blocks 10" x 12" x 2'6"
11'10"
3'7"
NOTE:
Open holes, 13/16" d. unless noted
1 Block 6" x 10" x 2'6"
2 Blocks 8" x 6" x 4'6" White Oak notched to fit in girder.
14'1½"
3'10½"
Notch blocking to clear strut L
STRENGTHENING
WASHINGTON BRIDGE over
HOUSATONIC RIVER
between MILFORD and
STRATFORD, CONN.
CONNECTICUT COMPANY,
BRIDGEPORT LINES
2 Pcs. 10" x 12" x 12'7½" H.P.
2-WB2
2-SC2
W2
C5
1 Pc. 10" x 12" x 11'6" H.P.
End of timber notched to fit in floor beam
Notch timbers to fit in caps
SC1
W1
1 Pc. 10" x 12" x 12'5" H.P.
L5
L6
Block 8" x 12" x 2'4 White Oak
36-¾" ϕ Bolts x 1'2"
2 Pcs. 10" x 12" x 2'4½" H.P.
2'4"
Cored holes
Finish
Make 2-Cast Iron Caps C5
Wedge finished over
Make 2-Forged Steel Wedges W2
13/16" d. Holes
Slots
4'6"
Make 4-L 8" x 10" x 15# x 4'6" S.C 2
Std. Rev. Nut
Std. U. S. Thread
Make 2-Bolts W 8.2

timber struts, and its ends butted against blocking on the channels of the vertical truss posts, blocking also being placed within the post to stiffen it locally.

In the full panel at each end of the span (Plate XV) the temporary $\frac{7}{8}$-in. rods were provided with turn-buckles for greater convenience, although adjustment was possible at the yokes where the main rods were connected with the U-bolts passing around the pins.

Specifications.

As is frequent in the writer's practice in the design and construction of such bridges or their repair, the specifications prepared by George F. Swain, Past-President, Am. Soc. C. E., and issued by the Massachusetts Railroad Commission, were adopted as the basic specifications for this work. As the proposed method of conducting the reconstruction required great care in its execution, or it would result in disaster instead of an improvement, supplementary specifications were prepared going into more than usual detail as to the methods to be used, precautions to be taken, and the order in which the work should be performed.

It was provided that the electric railway traffic should be suspended across the swing span during the progress of the repairs, and that other traffic should be restricted at all times to the half-width of roadway away from the truss on which work was being done at the time. In preparation for renewing the chord of the north truss, the sidewalk was to be removed and stored properly so that it might be replaced after the chord had been renewed.

The strengthening of one truss was required to be completed before work should begin on the other; and for each side of the bridge it was prescribed that the work should proceed in the following order:

The new longitudinal strut in the pivot panel under the truss between the cross-girders, should be placed and entirely riveted, except as noted, before any work is done on the truss itself.

Obstructions to placing the temporary chords, such as tie-plates and lattice bars, might be removed when necessary, but in such case temporary blocking and clamps should be placed in order properly to stiffen the chord channels.

All the temporary material for one truss was to be put in place and the slack taken up (but without strain), and examination was to be

made of each member to see if the anticipated additional wedging would leave the wedges within the prescribed limits of position, before the wedging should be begun, which was to relieve the main chords. The driving of these wedges was to begin at the pivot panel and proceed thence toward the ends, keeping the conditions approximately equal in the corresponding panels of the two arms. Wedges were to be well lubricated, and when fully drawn into the position which should remove all the strain from the main chords, they should bear for at least three-quarters of their length on the caps. All the bolts in the scab channels for the temporary struts were to be put in place and the nuts placed on the ends of the bolts, but left loose until the wedges were in their final positions. After the wedges were adjusted so that no strain whatever was left in the main chords the nuts must be tightened before any chord section could be removed.

It was specified that the entire temporary chord, including the adjustable rods of the end panels, should be in place and satisfactorily adjusted so that the main chord should be entirely free from strain, and the bolts through the scab channels tightened up, before any section of chord channel might be removed. The renewal of the chord channels should begin at the ends and proceed toward the pivot panel, only one section of chord channel being off at once, the lacing and tie-plates being bolted on as soon as practicable. Where the temporary work prevented immediate replacement of tie-plates and lacing, blocking and clamps should be used to stiffen the new chord channels similar to those specified for use with the old ones before renewal.

Wedges on the cross-girders under the pins, L_6, were specified to be driven before the old chord was removed, and to be withdrawn after the new chord sections should be in place; after which they were to be redriven to a firm bearing and secured in such a manner as to prevent their working out. At the time of writing the specifications it was not realized that the pin-holes in the chord channels had been so deformed by rust and overstrain that a settlement of from $\frac{1}{8}$ to $\frac{5}{16}$ in. had occurred at these points, requiring a corresponding lift of the whole bridge in order to allow the new chord channels to be put on.

The experience in driving the wedges the first time led to the cancellation of the requirement for their withdrawal and redriving. It was specified that short pieces of the inside channels at L_6 (there being

four channels in each chord section in the pivot panel) should be left in place to help deliver the weight of the bridge to the cross-girders.

Precautions Taken to Insure Safety.

From the inception of this method of renewing the truss chords, it was fully appreciated that any carelessness, either in design or execution of the work, would be fraught with danger; so special precautions were taken to insure its success. Consideration was given to the matter of economy, but when the question was the choice between low cost attended by danger and a somewhat higher cost coupled with safety for life and material, there was no hesitation in choosing safety. In its basic principles, the method proposed was so decidedly economical that there was less incentive to reduce the cost by the last cent possible, or to use too delicate scales in balancing safety against cost.

Extensive experience in the conduct of repair work brought a realization that, even more than in new work, carefully prepared specifications and plans were not by any means a guaranty that the actual work would be done as indicated by them. Therefore, when the plans were presented, it was stipulated that, if they were adopted, the work must be done under careful engineering supervision by a contractor, foreman, and crew who had been found dependable on other similar undertakings.

It was recommended, also, that payment for the work should be put on such a basis that undue desire for profit should not lead the contractor into careless methods. Arrangements were made to give the writer absolute jurisdiction, and that he should be represented at all times on the work by a trusted assistant as resident engineer; also, that he should be present in person during the more critical stages of the work. A contractor and foreman who had aided the writer and his staff many times in seeking "safety first" on other critical work, and had inspired complete confidence, were chosen to do the work, which was carried on under a "force account" contract.

In many cases the stresses to be provided for would have permitted the use of smaller timbers, and correspondingly smaller wedges, caps, etc., for the temporary work, especially if it could be assured that they would be subjected to no other strain than that of direct compression parallel with the axis of the timber. Practical consideration of all the demands that might be made upon them, however, prompted the use

of 10 by 12-in. timbers throughout, and wedge and channel reinforcement of a uniform type, as far as practicable.

As a further precaution, extra timbers, wedge, caps, and bolts were provided for use in an emergency, and timbers and blocking were made available so that, in case of difficulty in drawing up the wedges, the bridge could be swung open and blocked up on the fender pier. Except two wedge bolts, these extra supplies were not required, and after the completion of the work the timbers were returned to the lumber company without other expense than the mere hauling to and from the bridge.

Although it was considered safe to swing the draw at any time during the progress of the work, it was thought desirable as an additional precaution to avoid such turning if practicable. Preparation was made, therefore, to execute the work during the season in which navigation would be closed by ice. It is evident that the writer can lay no claim to being righteous, for "the good book" states that the "fervent prayer of a righteous man availeth much." Certainly his prayer for a long closed season was fervent; but the winter of 1912-13 proved to be a very unusual one, and the longed-for ice came in such moderation that navigation continued without interruption. Nevertheless, the work went ahead and was completed with no mishap whatever.

Construction Work.

All materials for the work, both for temporary use and for permanent construction, were made ready and collected on or near the bridge before work was commenced on the existing structure. A barricade was erected at the end of each approach span, just outside of the permanent gate, and a fence was built for the whole length of the swing span to confine the traffic to the south side of the draw, leaving a little more than half the width of the roadway clear, and protecting the men at work on the north truss. The sidewalk floor outside the north truss was taken up, and the north tender's house (used only for storage of supplies) was removed except the roof, which was hung by slings from above because of the counter rods of the truss passing through it.

The first work, except that of preparation, was the removal of the longitudinal struts between the cross-girders of the turn-table and their replacement with new material. This was completed and riveted (except one diagonal omitted for the time being to allow the wedge

cap of the temporary chord to pass through the strut) before any work was done on the truss chord itself.

The temporary chord (or temporary substitute for the chord) was then put in place, but without strain, for the whole length of the north truss. Each sub-panel strut was handled in two lengths, the long timber with its wedge-cap and the jaws for the pin comprising one part, and the short timber and its wedge-cap with the channel scabs bolted on loosely forming the other. These two lengths were put together in place, the bolts put through the scab channels and the long timber, and the nuts caught on the bolts, but not turned up (see Fig. 6).

This held the strut (minus the wedge) sufficiently in position to release the men and the tackle for the heavier work of hoisting the wedge into place, after which the bolts and washer plates were added and the wedge was drawn up just enough to take up the slack. The lengths were made such that this condition left the wedge bearing for about half of its length on about half of the depth of the strut, as shown by Fig. 7.

In the pivot panel, the double strut was placed in a similar way, with one line of timbers inside and one outside (see Fig. 8) of the permanent strut between the cross-girders, except that the wedge-caps (being double caps serving both struts and passing through the permanent strut where the diagonal was omitted temporarily) were placed separately from the timbers. Consequently, the scab channels were also handled separately. This was done the more readily here because these timbers could be hung upon the permament strut. In this panel the channel scabs were on the top and bottom of the timbers, and the wedges were drawn horizontally from the inside and outside, a single pair of rods serving for both wedges.

Before placing the upper temporary strut of the pivot panel, the §-in. wedges, W_3, were driven under the main truss pins between the pins, L_6 and the cross-girders (or bearing girders). As these pin-holes had been enlarged by rust and overstrain, there had been such a settlement as to require a lift ranging from $\frac{1}{8}$ to $\frac{5}{16}$ in. The cramped space for these wedges added to the difficulty of driving them into place, but this was accomplished satisfactorily. Afterward, the upper temporary strut was placed between the truss verticals. The wedges under these pins were left in place, not being withdrawn temporarily as specified. It

was not found necessary to pin them to prevent their working out. Blocking was used inside the posts, and between the strut timbers and the posts, to distribute the strain and stiffen up the post channels. In this temporary strut the scab channels were placed on the upper and lower sides of the strut, and the wedge was driven horizontally.

Temporary tension rods, TR_1, in the end panels (see Plate XV) were placed by first inserting the long rods through the lacing of the chord and passing their ends through the pin-caps of the temporary struts. Yokes were then put on the ends of the rods and the nuts turned up, the loop ends put around the pins and through the yokes, and the turnbuckles added and turned up sufficiently to take up the slack.

In placing the temporary struts or rods, it frequently became necessary to cut away the tie-plates and lacing bars, but whenever this was done suitable blocking and clamps were put on to stiffen the chord channels properly (see Fig. 9). This was done also to stiffen the channels of the new chords before the temporary struts were removed, so that the tie-plates and lacing could be bolted on. At some pins, a slight readjustment of the pin-packing was required to allow the proper insertion of the jaws of the temporary struts.

When the temporary work for the renewal of the chord of the north truss was in place for the whole length of the swing span, with the slack taken up, but without taking any strain, all parts were examined carefully to see if any refitting was required. Everything being found satisfactory, the work of drawing in the wedges was begun, starting at the pivot panel and working toward the ends, keeping the advance approximately even in the two arms. At each wedge the nuts were turned up a little on the rods alternately, using a long-handled wrench with the handle lengthened by the addition of a piece of pipe so that three men could work together on the wrench while a fourth assisted by striking the head of the wedge frequent blows with a sledge. The chord was considerably relieved of strain in the pivot panel, L_6-L_6' (see Fig. 5), and the adjacent panels, L_4-L_5-L_6, toward each end before any wedging was done in more distant panels. The middle panels, L_2-L_3-L_4, of each arm were then wedged until there was hardly any strain left in the truss chord. The men returned to the pivot panel and those next to it on each end, and the strain was entirely taken out of the chord. After this, the remaining strain in the middle panels was removed, and the end panels, L_0-L_1-L_2, were relieved of all possibility of strain,

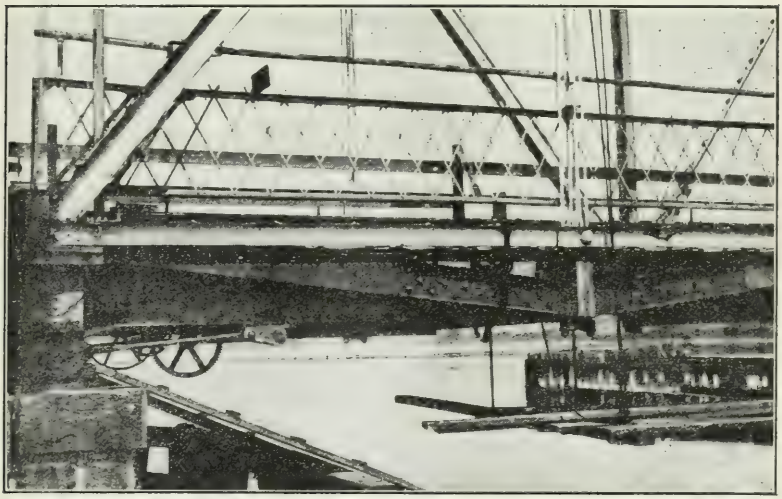


FIG. 6.—TEMPORARY CHORD IN PLACE AT WEST END OF SOUTH TRUSS.

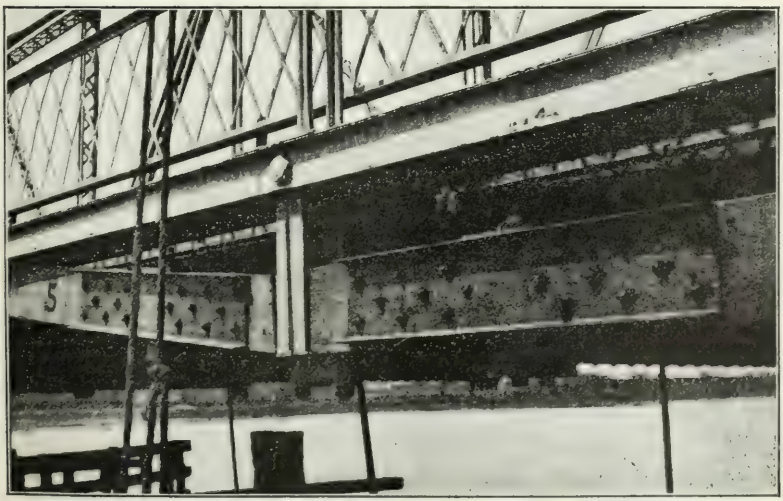


FIG. 7.—TEMPORARY BOTTOM CHORD, L_4-L_5-L_6, IN SOUTH TRUSS.

whether the ends of the span were lifted or were swinging free, by wedging the compression struts and by tightening up the turn-buckles on the tension rods.

As had been expected, a slight strain was now found in the chord of the pivot panel and those adjacent to it, although they had been previously entirely free from strain. Accordingly, the wedges in these panels were drawn in a little more to relieve the chord again entirely. Then each section of channel for the whole of the span was sounded carefully to make sure that it was absolutely "dead", and every nut on the wedge rods and each tension rod in the end panels was tested in order to be sure that the rods of each couple were dividing the stress about equally.

When everything was found to be in proper condition, the nuts were turned up tight on the bolts through the channel scabs, and the work was left for the night with a feeling of satisfaction. All pin-nuts for this truss were loosened so as to be removed easily and quickly (in most cases by hand) when required, but left on the pins, and so close to the chord channels that no channel could slip off the shoulder of the pin before its removal was desired. Several pin-nuts were broken in their removal, but a supply had been provided for this contingency. As more new nuts were required for the north truss than had been anticipated, an additional reserve was obtained in order that there should be no lack later.

The first work the next morning was to retest carefully all details to see if any change had taken place during the night. Everything being satisfactory, the lacing and tie-plates were removed from the outer channel of the first panel, L_0-L_1-L_2, at the east end, and this channel taken off. The test of the plan of renewal was not to be delayed greatly; for hardly had this channel been removed when a whistle was heard calling for the opening of the draw. There was no time for substituting a new channel, so the work was stopped, all staging connecting the moving with the fixed structure was shifted to be carried entirely by the one or the other, and orders were given to open the draw.

During the whole time the bridge was open, the engineers and the foreman carefully observed the temporary structure, sounding the steel chord channels to see that they received no strain because of the movement. The result of this observation was so reassuring that thereafter no hesitation whatever was felt about opening the bridge as often

as desired; in fact, after the draw had been swung a few times, the work went on while the bridge was open just about the same as when it was closed, except for the removal of planks and ropes connecting it with the piers and fixed spans. Several times some of the workmen did not even notice that the bridge had moved until they heard the noise of the machinery lifting the ends after the bridge was closed.

When the draw was closed again, the new outer chord channel was put in place, and the hanger for the end floor-beam bolted to it. Then the inner channel was removed and a new one substituted. When both new channels of this section of the chord were in position and the nuts replaced on the truss pins, as many of the tie-plates and lacing bars as practicable were bolted on, and such blocking and clamps added as were necessary properly to stiffen that portion of the length which was left disconnected. Bolts were put in the gussets connecting the chord with the end floor-beam; the latter bolts were replaced by rivets as soon as practicable, but those in the tie-plates and lacing were allowed to await a more convenient time.

The renewal of the second and third panels followed rapidly, completing the east arm of this truss except for a short space near the pin, L_6. In like manner, the lower chord of the west arm of this north truss was renewed for an equal length; after which the new outside channels of the pivot panel were placed and the inside channels of this panel were cut off a few inches from the pin. As required by the specifications, a short piece of each of these inside channels was left in place to assist in delivering the load of the truss from the pin to the cross-girder.

As fast as sections of chord were added, the splice-plates were bolted tight with bolts in about half of the holes and drift-pins in the other half. Riveting of the splices was done as soon as convenient after the temporary work was removed, but riveting of the tie-plates and lacing, as well as the replacing of the sidewalk, was allowed to await the convenience of the work.

When the whole chord was in place and the splices bolted, all anxiety about the success of the plan was over, but some difficulty was experienced in the removal of the wedges. As their slopes were slightly less than the angle of friction of the bearing surfaces after the lubricant had been squeezed and dried out, some coaxing was required in starting them. In fact, in some cases it proved harder to force them out than it had been to draw them into place. It was considered unsafe

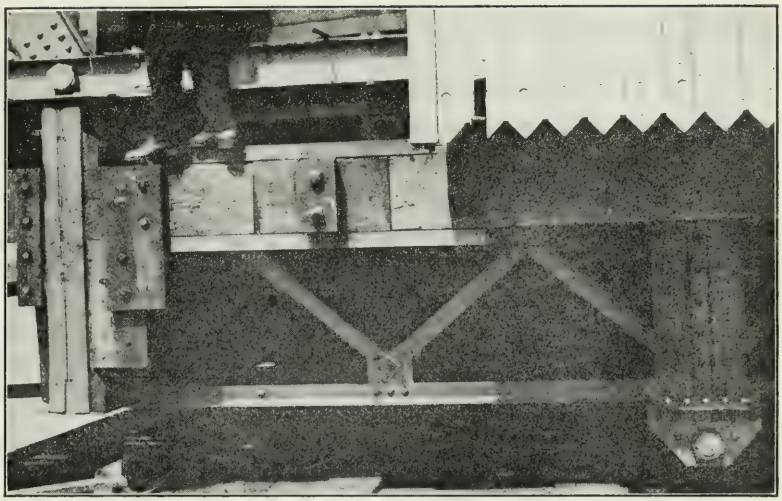


FIG. 8.—TEMPORARY BOTTOM CHORD IN PIVOT PANEL OF SOUTH TRUSS.

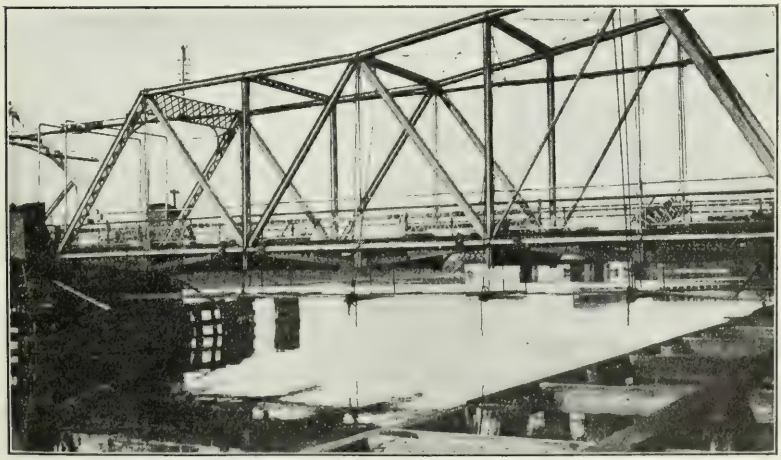


FIG. 9.—WEST HALF OF SOUTH TRUSS WITH TEMPORARY CHORD IN PLACE.

to remove both bolts entirely from a wedge, because a sudden movement of the wedge might endanger the chords by the impact as well as cause the loss of the wedge. The bolts were slackened a little and the wedge "teased" along by blows of a hammer against the side of the broad end, where it extended beyond the under side of the strut. In the worst cases, one of the bolts was slackened off and the other removed entirely, allowing a bar to be put through the hole in the washer plate and the wedge to be driven out by blows against the thin end of the wedge.

Work on the south truss was similar to that on the north truss with two important exceptions, namely, all the men were experienced in their respective duties, and the inconvenience of the sidewalk brackets, etc., was absent; progress therefore was more rapid. In one respect, however, work on the south truss was more difficult, because there was greater distortion and corrosion of the pin-holes at L_6 over the cross-girders. This caused the most troublesome operation on the whole work, as it added to the difficulty of raising the truss to its original position. The identical material was used for temporary work for the south truss that had been used for the north, except that two new bolts were required for drawing in the wedges, to replace those with worn threads.

After the new chords were in position, riveting and some adjustments of floor-beam hangers completed the restoration of the bridge to a capacity considerably greater than before corrosion and distortion had injured it.

Conclusion.

In comparison with what would have been required had the repairs been made by providing for a detour, so that the draw span might be blocked up on the fender pier for the renewal of the chords, the money cost and the interference with traffic by this method of reconstruction were remarkably small.

The total cost of the work, exclusive of engineering expense and that of watchmen, was $5 900. Vehicular traffic was confined to the half-width of bridge for about six weeks; trolley passengers were obliged to transfer, with a walk of about 300 ft., for a similar period. Shipping suffered no obstruction whatever, for the bridge was swung as freely as usual.

Although no definite estimate was made of the cost of a detour, with a temporary viaduct and jack-knife draw, it is believed that this

cost, with provision for car service, would have been at least $100 000. This plan would have saved the transfer of trolley passengers, but all traffic would have had an increased length of haul of 300 ft. (about equal to the distance that traffic was restricted by the adopted plan), and an undesirable grade at each end of the detour would have been introduced. As a temporary draw is slow of action, the interruption to traffic probably would have been as great as it was with the short gauntlet required by the plan described in this paper.

Interference with navigation surely would have been greater with a temporary trestle and jack-knife draw, unless there was justification for the suspicion several times expressed when a tug passed up the river a short distance, and returned soon without any apparent reason for the trip. It was intimated that these trips might have been taken in the hope of a refusal to open the draw, in order to obtain a basis for a damage claim. No cause for such claim was given, however. The time required for the work would have been much greater with such a detour than was the case by the method adopted.

The cost of a detour might have been reduced somewhat by omitting provision for the use of cars on the temporary bridge, but even with this saving, it would have involved many times the cost of the method adopted. Such an omission would have required a transfer of trolley passengers and necessitated their walking about ¼ mile.

Although the success of the method was never doubted by the writer, he realized that some individual might be criminally careless and neglect his plain duty, or that the seriousness of some otherwise minor accident might have been increased because of the reconstruction work. There was therefore a feeling of relief when the work was safely completed, also intensified by the fact that an eminent engineer had expressed a fear that "the bridge would be dropped into the river before the task was finished."

Hearty appreciation is felt for the constant interest shown by James L. Parker, Assoc. M. Am. Soc. C. E., the writer's Principal Assistant, and Guy Pinner, Assoc. M. Am. Soc. C. E., his Detail Designer and Resident Engineer during construction, for their careful study of the problems involved; also to the United Construction Company, the contractor, and to Mr. Charles Sones, their faithful and careful foreman, and to each of his men for their helpful attitude, so necessary for the successful prosecution of the work without the slightest accident.

DISCUSSION

Mr. Thomson.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—The author is to be congratulated on the wonderful ingenuity shown, the marvelous economy of construction, and the successful manner in which this work was carried out. Only those familiar with this kind of work can realize how many opportunities there are for careless assistants to drop the bridge in the river, as in the case of the Bussey Bridge, many years ago, and others.

The speaker would like to ask Mr. Keith whether or not he considered reinforcing the bottom chord with concrete instead of removing it? It hardly seems possible that any cheaper method could have been found than the one adopted.

Mr. Pinner.

GUY PINNER,* ASSOC. M. AM. SOC. C. E.—Little can be added to this excellent paper, except that Mr. Thomson's question as to whether strengthening was considered might be answered.

When the problem first came to the speaker's attention in Mr. Keith's office, it was thought that the old chords could be strengthened by adding section in some manner, but, after a thorough investigation, it was found that there was not enough section left in the old channels on which to begin work, so the thought of strengthening was abandoned in favor of entire renewal.

The author lays considerable stress on the matter of "safety first", which was always kept in mind in starting all details of the construction, as well as in making the plans for the work. When this work was started, it was thought advisable to test the "safety first" ideas of the foreman whom the author has mentioned, and, therefore, when the scaffold hangers, which were made by a local blacksmith, were received on the work, several of them were tested and found to be satisfactory and about as the calculations indicated.

When the erection of the new lower struts between the cross-girders was begun, the foreman projected several 3 by 8-in. or 3 by 10-in. timbers from the upper longitudinal truss struts in the center panel for swinging them into place. They did not appear to be strong enough to carry the weight of the struts, but calculations showed that the timbers would have an extreme fiber stress of about 1 400 lb. per sq. in. After these tests, the conclusion was reached that the foreman could be relied on with regard to safety, which conclusion was proved later, as the work was completed without an accident.

The gang was a very good one for this class of work, each man being a hard worker, thoroughly dependable, and following all directions to the letter. Although the work was explained to the men several times, they never seemed to grasp the intent and importance of it,

* New York City.

Mr. Pinner. until probably the last week, after all the temporary chords had been removed. They had discussed it frequently among themselves, but could not understand why it was necessary to do so much tightening and adjusting on the bolts which passed through the adjustable wedges. When the work was finished, however, they stated that they would like to be sent on the next contract of this nature, for they felt that they could make a better job of it.

Mr. Hardesty. SHORTRIDGE HARDESTY,* ASSOC. M. AM. SOC. C. E.—It is instructive to note that Mr. Keith's plan of reconstruction gave direct central stresses in all the temporary struts; also that, with one exception, these struts bore centrally on the truss pins or abutted squarely opposite each other, thus avoiding bending or twisting stresses in other truss members. It was found advisable to put the strut, $TC6$, 1 ft. 1 in. above the pins at L_6, thus throwing bending into the post, $U_6 L_6$. This bending was small, however, being merely that due to the horizontal component of $U_5 L_6$. The author's solution, therefore, was not only a very practical one, but was theoretically correct as well. The method of bringing the new chord into action also insured that the stresses would be properly distributed.

It would seem that the eminent engineer who expressed the fear that "the bridge would be dropped into the river before the task was finished" had not examined the author's details and specifications very thoroughly. The bridge was probably as safe with the temporary wooden chord as with the old steel chord, which must have been near failure at some of the pin-holes.

Mr. French. JAMES B. FRENCH,† M. AM. SOC. C. E.—Work such as that described by Mr. Keith is not monumental. It leaves little behind to attract favorable attention, but, at the same time, it often calls for greater skill on the part of the engineer and draws on his resourcefulness and ingenuity to a greater extent than the execution of more monumental works which remain to be looked at and admired. This structure was, presumably, not any more beautiful after these repairs were made than it was before, and it is doubtful whether any of the people who were carried over it while the work was in progress had any appreciation of the care that was being taken to protect their lives.

It is obvious that the work described could only have been done as the result of careful investigation, careful planning, and careful supervision at every stage. Any mistake, either in the planning, execution, or supervision of the work, could easily have resulted in dropping the bridge into the river, as the author's friend prophesied would happen, and serious loss of life might have resulted.

Although the author's precaution to have all the material on hand and an extra supply of tools available before the work was started

* Kansas City, Mo.

† New York City.

Mr. French

might not seem worthy of note, the importance of such precautions in cases where human lives are concerned cannot be exaggerated. The speaker has had many occasions to be thankful for the training received years ago under a very capable practical bridge erector in rebuilding railroad bridges under traffic. Great pressure was sometimes brought to bear on this man to make a spectacular start before he was thoroughly ready, but under no circumstances would he in any way weaken the old structure until he had positive first-hand knowledge that all the materials and tools required for the completion of the new one were just where he could put his hand on them. Such schooling is not without value to any one responsible for operations of the kind described in this paper.

The character of the contract entered into for the execution of this work also seems worthy of comment. It is, of course, an axiom in the writing of specifications that it is permissible either to specify the result to be obtained, allowing the contractor full discretion in the choice of the methods to be used and holding him fully responsible for the result; or to specify in detail the method and procedure to be followed at every stage, with the understanding that the contractor is thereby relieved of responsibility for the result. Making a contract of the first sort in a case like the one described by Mr. Keith has more than once led to disastrous results, and considerations of public safety make such practice particularly dangerous unless bidding can be confined strictly to contractors known to be thoroughly competent and to have conscientious regard for the public safety. Conspicuous examples illustrating the disastrous consequences of putting public work of this character under contracts which practically invite the contractor to take chances, will probably come to the minds of many readers of this paper, and such disregard for the safety of the traveling public should be condemned.

It has been the speaker's experience that railroad companies, which certainly receive their share of public criticism, are generally much more careful to take adequate precautions to protect human life in the execution of work of this kind, than municipalities. This experience has been gained largely in grade separation work for the Long Island Railroad Company, in Brooklyn, N. Y., and it is a pleasure to testify that in all important situations involving the protection of human life, this company has gone to the expense of having plans and specifications prepared, not only to cover the permanent construction, but the temporary structures and falsework, also carefully worked out programmes of procedure to protect the traveling public at every stage of the work.

In order, however, not to hamper the ingenuity or resourcefulness of the contractor, or to relieve him of responsibility, these plans and specifications relating to falsework and procedure have not been in-

Mr. French.

sisted on as final, but the contractor has been given the alternative either of following them or submitting others, equally definite, for approval. In no case, however, was the particular work allowed to proceed until the contractor and the engineers of the railroad company had come to a definite understanding as to how it was to be handled in detail. In this way, contractors and engineers have shared the responsibility, and the public has received the most effective protection practicable.

The author has apparently been employed by a corporation having the same high regard for its obligation to the traveling public and, in the speaker's opinion, both he and the corporation are to be congratulated on the very skilful and economical execution of a particularly difficult piece of work.

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Paper No. 1449

PHYSIOGRAPHY OF WATER-SHEDS AND CHANNELS, AND ANALYSIS OF STREAM ACTION OF SOUTHERN CALIFORNIA RIVERS, WITH REFERENCE TO THE PROBLEMS OF FLOOD CONTROL *

By A. L. Sonderegger, M. Am. Soc. C. E.

With Discussion by Messrs. Charles A. Pohl, Francis H. Wright, H. F. Dunham, Fred D. Bowlus, and A. L. Sonderegger.

Synopsis

Although the basic forces which have brought about destruction by flood flow, and, in consequence, the necessity for flood control in Southern California, are the same as in the Eastern and Northern States, they operate in this country under widely different conditions with the result that their effect is intensified.

The chief physiographic features of Southern California are the result of block-faulting, the mountains being raised blocks and the valleys sunken blocks. During the process of deformation, materials were eroded from the mountain ranges and transported by water into the depressions or sunken valleys, producing a valley fill made up of gravels, sands, and clay. Apparently, no glacial action has occurred in Southern California.

The sunken valleys present basins of enormous dimensions, embracing areas up to 100 sq. miles, and depths of more than 1 000 ft., and

* This paper was presented before the Southern California Association of Members of the American Society of Civil Engineers, at its meeting of March 12th, 1919, and also before the meeting of the Society on October 1st, 1919.

they are the logical storage reservoirs for the flood waters. The velocity of percolation of ground-water is exceedingly low, ranging from ½ mile to 4 miles per annum, the water remaining fresh and potable for indefinite periods. There are some surface storage reservoir sites in the San Jacinto block and in the San Bernardino block, but there is none of any consequence in the San Gabriel Range.

In Southern California, 75% of the precipitation occurs in the months of December to March, inclusive, and limits flood discharge to those months. In summer the river channels are essentially dry. Flood flow, therefore, may begin to act on a dry river bed, which, subsequently, becomes saturated and is finally exposed to a protracted receding stream flow until it dries up.

The short courses of the rivers from their source, and more particularly from the mouth of their canyons, to the sea, traversing an alluvial formation on steep grades, produce below the mouth of the canyons a rapid succession of alluvial and erosional stream activities superimposed upon the general phenomenon of valley construction. The effect is an alternation of stretches where the stream inundates the adjoining country, with sections where it is in a deep cut and undermines the banks.

The most important law affecting stream flow in a natural channel crossing alluvial deposits which readily succumb to erosion, is expressed as follows: The stream has a tendency to charge itself with a burden which is in exact proportion to the power of the water to carry. With constant discharge this power or capacity to carry a burden varies on an average with the fourth power of the mean velocity (Gilbert), or with the fifth power of the bed velocity (Deacon).

The profile of a stream shows a gradually flattening gradient from the source to the sea. (See Fig. 1.)

Every one of the larger streams shows the following characteristic sections: First, a mountain section extending from the source of the stream to the mouth of its canyon. It has the steepest grades and highest velocities, and is the original source of all débris. There are no agricultural or other improvements of any consequence in the mountains, and there is no damage from flood flow.

Second, a débris cone, defined as extending from the mouth of the canyon as far down stream as action is alluvial. (See Fig. 2.) Erosional activity and the transportation of detrital matter

in the mountains being the cause, the formation of the débris cone is the effect. The cone is a fan-shaped area on which the river deposits the larger portion of its débris in traction, and only a little of that in suspension. The stream course is along lines of steepest gradient, issuing from the mouth of the canyon. The danger lies in a shifting at the mouth of the canyon, causing the stream to pursue an entirely new course. The cone offers the most favorable opportunity for the absorption of flood water. The muddy water, during peaks, prevents seepage, and present methods of artificial spreading have not succeeded in reducing the peak flow.

For the San Gabriel River cone it has been proposed to deepen the existing channel and construct embankments of the excavated materials, leaving wide berms. This channel is to be maintained by dredging whenever necessary.

Third, a valley section extending from the foot of the débris cone to the head of the delta, with a channel alternately under erosional and alluvial stream action. The effect of erosion on a channel during the peak flow is a scouring of the bottom, and during a receding flood an undermining of banks, causing the liberation of large quantities of débris and their movement down stream. (See Fig. 3.) The damage is caused by the undermining of the banks. Under alluvial action the stream runs in a shallow channel, depositing a portion of its sediments both in traction and in suspension, and inundating the adjoining country. (See Figs. 4, 5, and 6.) Present methods of improvement are bank protection in erosional channels, and levee construction in alluvial ones, no attempt being made to check erosion of the channel floor.

Fourth, a delta extending from the last erosional channel of the valley section to tide-water. (See Fig. 7.) This is a typical deposit cone, the stream shifting more frequently than on a débris cone. Damage is caused by inundation. Coming to the sea beach there is a pronounced littoral current which tends to deflect estuaries southeastward. (See Fig. 8.) In the larger streams the tidal sloughs and lagoons at their mouths furnish tidal prisms of sufficient magnitude to keep their estuaries open the year round. With the smaller streams ocean breaker action closes the estuaries between flood seasons. The methods of protection contemplated for the Rio Hondo-Los Angeles River are: Levee construction along the course of deepest gradient, and rock jetties to stabilize the estuary.

The magnitude of the flood flow is the basic trouble; but in the absence of suitable storage sites it may be impossible, in many instances, to reduce either the peak or the average flood flow. The local problem resolves itself into one of preventing both erosion and movement of detrital matter.

Theoretically speaking, effective control of floods implies the following ten requirements:

1. That the stream arrive at the mouth of the canyon free from débris in traction.
2. That the stream arrive at the mouth of the canyon delivered of a large percentage of its sediments in suspension.
3. That it be prevented from resuming erosional action on its way down the cone.
4. That means be provided to induce the absorption on the débris cone of the largest possible portion of the flood flow, particularly of the peak load.
5. That erosional stream action be eliminated in the valley section by fixing the channel floor and protecting the banks.
6. That inundation areas in the valley section be defined and controlled.
7. That the stream arrive at the delta delivered of the major portion of its débris, both in traction and in suspension.
8. That inundation areas in the delta be defined and controlled.
9. That the estuary be made secure against closure through ocean wave or current action.
10. In each case exceptional as well as small flood discharge must be taken into consideration, and particularly the cumulative effect of exclusively moderate or small flood flow over a series of years.

General Conditions.

The characteristic physical features of Southern California are a coast line running roughly in a southeast direction and a series of mountain ranges running parallel to the coast line, separated by valleys made up of alluvial fill from these mountains.

The great variation in climatic conditions produced by these mountain ranges, roughly speaking, divides Southern California into three zones, in which the problems of flood control take on different

aspects. The first zone comprises Ventura, Los Angeles, San Bernardino, Riverside, and Orange Counties (with the exception of their desert areas). The ranges attain elevations up to 10 800 ft., and there is a characteristic seasonal rainfall in the valleys, roughly estimated at 16 in. The second zone embraces San Diego County, exclusive of its desert area, with mountain ranges rising to 6 500 ft., and a typical valley rainfall of about 14 in. The third zone includes the desert territory of the Mohave, Coachella, and Imperial Valleys.

Since the question of flood control is more acute in the northern part of Southern California, this paper will be confined chiefly to the first of these zones, and only touch the others; however, it should not be understood that there are no flood-control problems in San Diego County, nor in parts of the desert.

Geology.

The chief physical features of Southern California are the result of block-faulting. (See Plate XVII.) Generally speaking, the crustal movements which ultimately produced the present aspect of the country, raised the mountains, and simultaneously therewith the valleys between the mountain ranges sank. While this process was going on, materials were eroded from the mountain ranges and transported into these depressions, producing the valley fill as it appears to-day, so that the valleys and the coastal plain are not valleys of erosion but of construction. At the end of the Mesozoic period this part of the Pacific Coast was characterized by mountain ranges consisting of metamorphosed sediments, such as schists and gneisses, with intrusions of granite. During the Tertiary period, destructive erosion reduced these ranges to a very material extent, producing on land unstratified deposits of alluvial materials, such as boulders, gravel, sand, and clay, and in the ocean roughly stratified beds of conglomerates and more thinly stratified beds of fine sediments and clay materials, which later gave rise to the formation of sandstones and shales.

At the end of the Tertiary period the country was one of comparatively low relief. The elevation of the coast, as compared with its present elevation, was at least 1 800 ft. lower, and the sea beat against the San Gabriel Mountains or Sierra Madre Range.

At the end of the Tertiary period and the beginning of the Quaternary period, the coast was raised to not less than 700 ft. higher

than it is at present, followed by gradual subsidence to its present elevation.

The mountain ranges rise to nearly 11 000 ft., while the valleys, in general, slope from an elevation of 1 000 ft., more or less, down to tide-water. The valleys present a series of sunken basins or reservoirs which, in some instances, are practically completely surrounded by impervious formation, while in others they are open toward tide-water. Many of these basins are of a depth of 1 000 ft. or more, and cover large areas. The San Bernardino River basin, for example, has an area of more than 100 sq. miles and a depth exceeding 1 000 ft., bed-rock being at unknown depth. The basins, therefore, have enormous water storage capacities and become the natural recipients in their porous formation of large volumes of the rainfall running off the steep mountain slopes. Nature has thus provided storage for the flood waters of a magnitude compared with which the contents of the few artificial reservoir sites sink into utter insignificance.

In the northern counties of Southern California the porous beds of these valleys reach depths of 1 000 ft. or more, in fact, in many of them bed-rock has not yet been reached. This applies to the San Gabriel, the San Bernardino, the Pomona and the San Fernando Valleys. In San Diego County a different condition prevails. There, the valley fills are much shallower, reaching from 60 to 100 ft., and hence their storage capacity is quite limited.

As previously stated, block-faulting has been the chief agency in forming the present physiographic features. The most noted fault line is that of the San Andreas, which runs from Coachella Valley to San Bernardino Valley, and along the southern escarpment of the San Bernardino block northwest, along the northern margin of the San Gabriel block, toward the Tehachapi. (See Plate XVII.)

The San Bernardino block is tilted to the north, toward the desert, its steepest escarpment being to the south, just north of the City of San Bernardino. The top of this block is comparatively level, with numerous valleys and flats, and quite a number of good reservoir sites. Geologists tell us that these valleys are the remnant of the old topography, as it existed during the Tertiary period before the uplift.

The San Jacinto block is tilted to the west. Its steep escarpment is to the east, toward Coachella Desert, while to the west the slope is less abrupt. The eastern escarpment has a drop of as much as 10 300

PLATE XVII.
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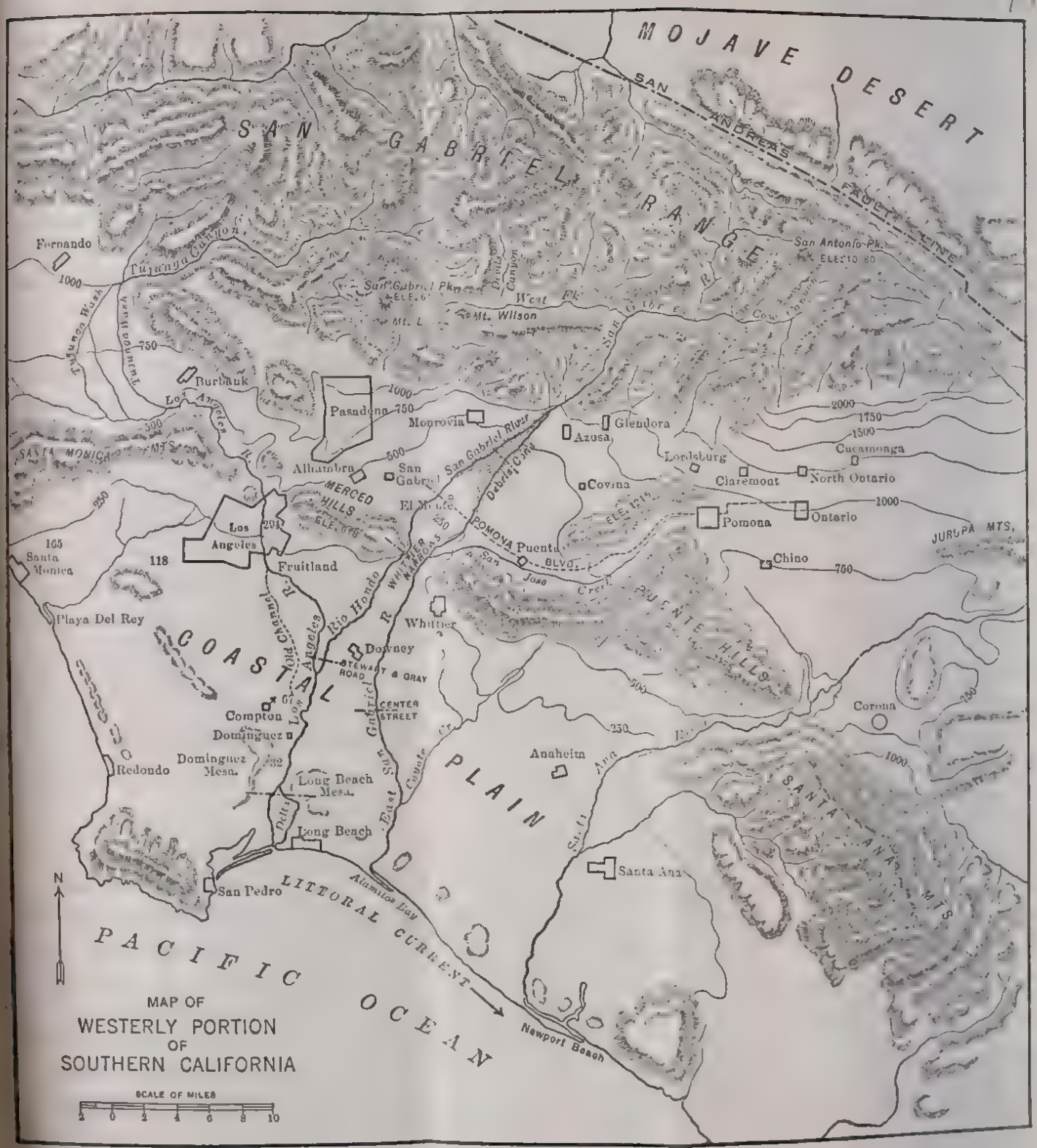
SONDEREGGER ON
PHYSIOGRAPHY OF WATER-SHEDS
AND CHANNELS.



ft. in a distance of 8 miles, while to the west it slopes from San Jacinto peak (Elevation 10 800) to the City of San Jacinto, about 9 000 ft. in 13 miles. This block also shows remnants of the old topography and has some reservoir sites. It is said that Perris Valley to the west of it, with its cupola-shaped granite buttes, is typical of the Tertiary topography which has undergone but little change.

The San Gabriel Mountains are assumed to be older than the San Bernardino Mountains, and to have been longer subjected to erosion, with the result that the flats have practically disappeared, and there are no reservoir sites of any consequence.

The main ranges consist of eruptive rocks, such as granite and diorites, and metamorphosed rocks, such as schists and gneisses, while the outlying lower ranges are mostly sedimentary. The latter is also the case with the Santa Monica and Ventura Mountains.

Generally speaking, the rocks are much shattered, and only the older granites, as we find them in Perris Valley, and in the Jurupa Mountains, near Riverside, are suitable for quarrying.

The fact that the mountain ranges run in series, and parallel to the coast, and the manner in which the outlying ranges were uplifted, has brought about a phenomenon peculiar to Southern California, in that the rivers have cut numerous gaps in the outlying mountain ranges, producing "narrows" which are characteristic of all the larger streams, and are seen in the narrows of the Los Angeles River at Elysian Park; of the San Gabriel River at Whittier; of the Santa Ana River at Colton; and of the Los Angeles and San Gabriel Rivers combined at Dominguez.

Physiography.

There are two factors that put Los Angeles flood-control problems into a class of their own.

The first is of meteorological origin, in that 75% of the seasonal rainfall occurs during the months of December to March, inclusive, resulting in a wet and dry season. Floods occur only during the winter months, while in summer the stream beds are practically dry.

Most of the smaller rivers are dry below the upper portion of their débris cones, except during storms. Even with the largest rivers, the ordinary flow is insignificant as compared with the flood flow or peak load. The action of the stream is, therefore, at the beginning of the

first storm, on a practically dry channel, and this action is widely different from that which occurs after the bed becomes saturated. Again, as a result of the semi-tropical rains on the steep mountain slopes, storm run-off is extremely "flashy", and it may happen that a flood channel that has been dry for six months may be filled to overflow within a few days. Finally, there is the process of the slow drying up of the stream. A receding flow is almost invariably alluvial in its action on the bottom of its channel, but may be under-cutting its banks, or causing saturated banks to cave, so that the condition of the channel at the end of the rainy season is often quite different from that during the flood.

The second factor is the short distance from the mountain peaks to the ocean. The air line from San Antonio peak, the source of the East Fork of the San Gabriel River, with an elevation of 10 080 ft., to the ocean is 44 miles, and the air line from the extreme water-shed of the Santa Ana River, with a maximum elevation of 10 630 ft., to the ocean is 84 miles. The length of the San Gabriel channel from the foot of the San Gabriel Mountains to the ocean, is only about 35 miles, with a drop of 750 ft., or an average of 21 ft. per mile. The stream profiles show the well-known parabolic curve gradually flattening out down stream. (See Fig. 1.) The short run, particularly from the foot of the mountains to the ocean, has produced in the stream channels a succession of sections in which erosional and alluvial action alternate.

Since it is in the very nature of things that the remedies to be applied to a section where stream action is erosional, must be widely different from those where it is alluvial, it becomes fundamental that the natural flood channel be studied, particularly in regard to the nature of stream action.

An erosional stream flows in a more or less rectangular channel of favorable hydraulic properties, sometimes with well-developed berms. It has a tendency to lengthen its course by attacking its banks and producing meanders, thereby flattening its grade and tending to establish equilibrium between its erosional power and the resistance to erosion of the stream bed.

During a flood two distinct erosional actions take place. First, there is scour of the bottom proportionate to the volume of flood flow, and the materials thus set in motion are transported down stream. As

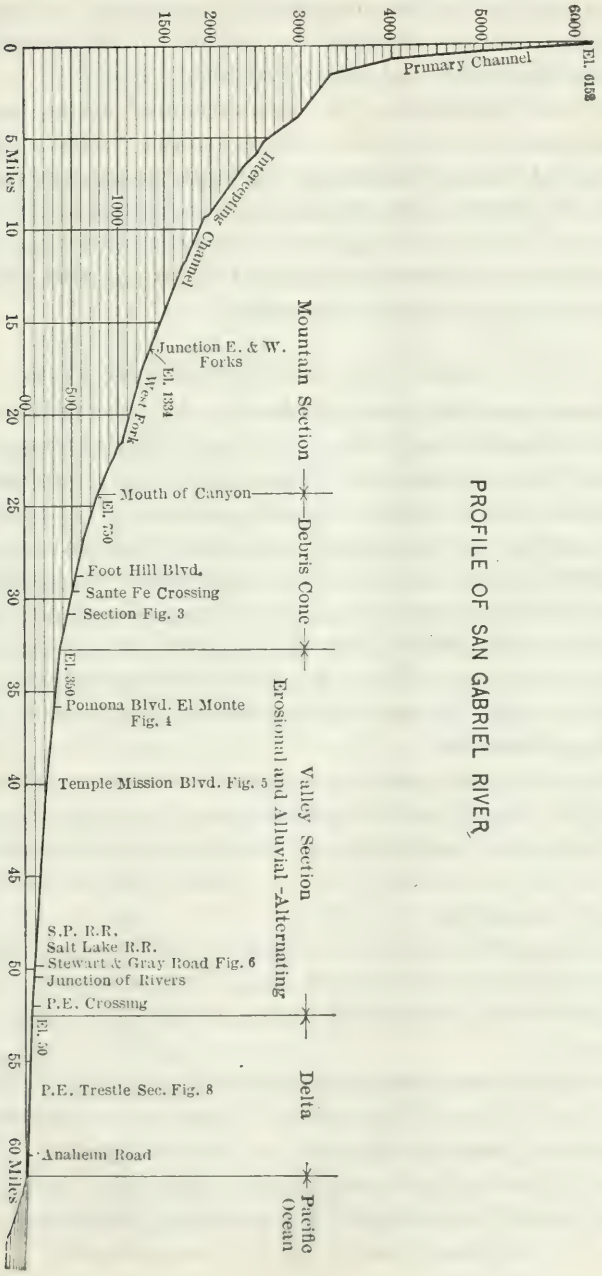

PROFILE OF SAN GABRIEL RIVER
Primary Channel
El. 6158
Intercepting Channel
Junction E. & W. Forks
El. 1334
West Fork
Mouth of Canyon
El. 750
Foot Hill Blvd.
Sante Fe Crossing
Section Fig. 3
El. 350
Pomona Blvd. El Monte Fig. 4
Temple Mission Blvd. Fig. 5
S.P. R.R.
Salt Lake R.R.
Stewart & Gray Road Fig. 6
Junction of Rivers
P.E. Crossing
El. 50
P.E. Trestle Sec. Fig. 8
Anaheim Road
Mountain Section
Debris Cone
Erosional and Alluvial -Alternating
Valley Section
Delta
Pacific Ocean
0
5 Miles
10
15
20
25
30
35
40
45
50
55
60 Miles
1000
1500
2000
3000
4000
5000
6000
500

Fig. 1.

the flood recedes, the bottom gradually rises again, and unless discharge drops off very suddenly, the bottom will be restored to the shape and elevation it had before the flood.*

The second phenomenon is the undermining of the banks, particularly along the outer banks of curves, and this results in a meandering course. At time of full channel the banks become saturated. With recession of the flood the external water pressure is withdrawn, and the banks cave, not only along bends, but also along straight sections. The falling earth is washed away and the damage done along banks increases with a protracted receding flood.

Fundamental Laws Governing Stream Flow.

The most important factor in stream flow is the relation of sediment to cross-section and slope. A stream running through an alluvial valley has a tendency to charge itself with a burden of sediment, which seeks to remain in exact proportion to the power of the water to carry it. It is, therefore, not enough to clarify a stream, but steps must be taken to prevent it from acquiring a new burden.

The scouring capacity of a river is supposed to vary with the square root of the velocity; but it is also very largely dependent on the depth of the flow. This is due to the increased weight. Shallow streams, as a rule, do not produce whirlpools and boils, while depth favors their formation.

Gilbert states that when the discharge is constant so that velocity changes with slope, then the capacity to carry a burden varies on an average with the fourth power of velocity, using the mean velocity instead of the bed velocity (near the bottom and which cannot be accurately measured).

Considering Gilbert's law, it becomes evident at once that even a small change in velocity must immediately affect the volume of burden carried to a considerable degree, and produce either scouring or a deposit.

The velocity itself is primarily dependent on the slope of the valley, and also on the shape of the channel. It varies in proportion to the square root of both the slope and the hydraulic radius. Width is a factor, a broad channel carrying more débris than a narrow one, but velocities and carrying capacity also increase with increasing depth.

* *Professional Paper No. 86*, U. S. Geological Survey.

If then a stream that is carrying its maximum burden changes its velocity, owing to a change of any of the numerous factors on which the latter depends, there must be a corresponding change of the combination of all these factors, and, in turn, a change in the volume of burden carried.

An important factor is found in the horizontal alignment of a channel, the carrying capacity being affected by turns and their curvature. The convex side of the stream suffers erosion, while the concave side, as a rule, shows the formation of a bank.

Tributaries produce a steeper slope after the point of confluence. An affluent brings in a quantity of material which the main stream, having its own burden to carry, cannot entirely absorb under the same grade. As a result, deposits may occur below the point of junction, until the river-bed is raised and the velocity increased to such an extent as to take care of all sediment. Upon last analysis it is found that the ruling element is that of the slope.

Careful study of the old river channels will often disclose the very important fact that, in almost every one of the differing sections, stretches may be found where the river is in a state of equilibrium, neither attacking banks, overflowing, nor depositing, and scouring its bottom only during extreme high floods. Particular attention should be paid to such portions of the river-bed, and all its properties ascertained, such as geological formation, character of the banks, slope of banks, their vegetation, grade of the river-bed, its cross-section, horizontal alignment, and the character of the débris. In the writer's opinion, these properties must form the basis for the solution of the problem at hand. Nature often will indicate what type of channel will best serve the purpose.

In the discussion which follows reference is made by way of example to the San Gabriel River as being a characteristic normal stream, and one of the most important in Southern California. It has a water-shed above the mouth of its canyon of 222 sq. miles, and a flood peak exceeding 30 000 sec-ft.

Every one of the larger rivers exhibits at least the following characteristics:

1. A mountain section, in which stream action is erosional, this section being defined as extending from the source of the stream to the mouth of the canyon.

2. A débris cone, extending from the mouth of the canyon as far as river action is alluvial.

3. A valley section, beginning at the foot of the débris cone, where river action becomes erosional. This district, as far as the tidal rivers are concerned, may be defined as extending to the delta, while with intercepting streams it would reach to the point of junction with the main stream. River action in the valley section, as a rule, starts with erosion. A few miles below, it may turn into alluvial action, then back to erosional, and again to alluvial. The alternation is apparently more frequent in the smaller than in the larger streams.

4. A delta section, which is the last alluvial sector, and terminates at tide-water.

Description of Characteristics of Individual Sections.

The Mountain Section.—The mountain ranges of Southern California rise from 1 000 to 10 000 ft. above sea level. The foot-hill region along the line where canyons debouch into the valleys, is about at an elevation of 1 000 ft. By far the greatest portion of the mountainous area is below Elevation 6 000. As far as the San Gabriel, San Bernardino, and San Jacinto blocks are concerned, the formation is granitic and metamorphic, including gneisses and schists. The outlying hills are either shale or old alluvial formation. An exception to this rule is found in the Santa Monica and Ventura Mountains, which are sedimentary.

As a rule the mountains are covered either with dense brush or timber. The lower timber line is at 5 000 ft. Invariably, it is found that slopes dipping to the north have a better forest cover than those more directly exposed to the rays of the sun. A shady slope shows a thicker crust of earth than a sunny one. The absence of a brush cover exposes the slope to washing, leaving often only a thin crust of earth overlying bed-rock.

The grades of the mountain sides are very steep, slopes of 2 000 ft. per mile being common; consequently, the stream beds of the side canyons, which may be called primary creeks, are equally steep. These primary streams generally flow on bed-rock, often along fault lines, and in narrow gorges. The action of the floods keeps these gorges well scoured. It is believed that the quantity of débris transported by individual primary canyons is comparatively small. The

writer has, however, no specific data to offer in support of this opinion, and reliable information on the subject is much needed.

The steep primary canyons, as a rule, discharge into a collecting channel of much flatter grade. Always a short distance below the point of a junction is to be found a deposit of débris, which is really the débris cone of the primary channel. Since primary channels, as a rule, follow in rapid succession, it is found that the intercepting channel presents a comparatively wide bed of deposits of large boulders, gravel, and sand along its entire length.

In order to make this description more tangible, reference is made to the West Fork of the San Gabriel River. This fork issues from San Gabriel peak, with an elevation of 6 152 ft., as a primary creek for about 1½ miles, and drops to an elevation of 3 500 ft., where it assumes the rôle of an intercepting channel.

The general course of the West Fork is easterly, intercepting numerous primary channels, and some minor intercepting channels from both the north and south sides. The length of the intercepting channel is about 14 miles, and the drop 2 200 ft., to the junction with the East Fork of the San Gabriel River, at an elevation of 1 360 ft. The primary tributaries intercepted by the West Fork have an average grade of 2 000 ft. to the mile, while the intercepting channel itself averages 160 ft. to the mile. From the junction of the West and East Forks to the mouth of the canyon at Azusa, the distance is 9 miles, and the drop about 600 ft. Although it is quite clear that the primary channels are under the constant action of erosive forces, it would appear that the secondary or intercepting channel remains more or less in a state of equilibrium. During each flood, primary channels will bring certain quantities of débris into the valley, and these, in turn, are carried on during the flood, with the result that, over a period of years, these intercepting channels remain about the same, showing neither permanent cutting nor alluvial action. This fact should be remembered in the discussion of the effect of check dams; because should the transportation of the coarse débris from a primary to an intercepting channel be stopped, the stream, when reaching the latter, would have a capacity to load itself with a new burden, and would begin erosion until a flatter grade had been formed and a new state of equilibrium established.

Attention is called to another fact—the comparatively short length and low drop of an intercepting channel, as compared with the sum of the lengths and drops of the primary channels which feed it. The difference is particularly striking in the drop, which may be only one-twentieth of that of the primary channels. These facts should receive attention in considering the feasibility of constructing check dams over an entire water-shed. If it is simply desired to arrest the transportation of débris, it might be done at less expense in the intercepting channel than in its primary tributaries.

The canyons, while all steep, do not all have the characteristics of highly erosional channels. Canyons filled with large deposits of alluvial materials, particularly of old alluvium, are frequent. These deposits are well consolidated, are covered with brush and timber, and apparently absorb a large proportion of the rainfall, even during periods of heavy freshets. They invariably produce, comparatively speaking, a prolific summer flow. Their stream beds indicate that flood discharge occurs in a moderate volume, as compared with canyons which may be immediately adjoining, and which have all the markings of wild storm channels. Apparently, in these canyons, Nature has produced what man is trying to accomplish, and for this reason they should be studied in solving problems of river improvement.

Summarizing the characteristics of the mountain section, it may be stated that it has the steepest grades, the most rapid run-off, the highest velocities, and last, but not least, it is the original source of all débris.

As the disintegration of rocks and the formation of débris cannot be checked, there remains the prevention of its transportation into the valley below.

The Débris Cone.—Below the mouth of a canyon, grades become flatter, and the coarser débris brought down from the mountain section is deposited in fan-shaped areas, commonly called débris cones, and here defined as extending from the mouth of the canyon as far down stream as action is alluvial. Erosional action and the transportation of detrital matter in the mountain section may be defined, therefore, as the "cause", and the formation of the débris cone as the "effect."

The channel on a débris cone is, as a rule, a pronounced cut, not always entirely straight, but not materially deviating from the line

of steepest grade. A slight shifting at the apex, however, may cause the stream to follow an entirely new path, although still along a line of steepest gradient. It is a fact that the San Gabriel River at Azusa might find a channel either side of its present course. In either case, it would invade a large area now in a high state of cultivation.

In the very nature of things, it is difficult, if not impossible, to maintain a stream on the débris cone permanently in the same channel. Fig. 2, compiled from data contained in the Los Angeles County Flood Control Report of January 2d, 1917, represents a cross-section through the débris cone 4 miles below Azusa.

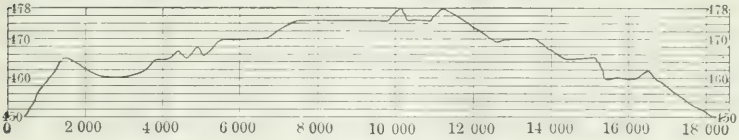


FIG. 2.

The cone covers a roughly triangular area, with its apex at the mouth of the canyon at Azusa at an elevation of 750 ft., and with a base about 3 miles wide at a distance of 6 miles below Azusa. The débris at the apex varies from large boulders, weighing several tons, to coarse gravels, while at the base the largest boulders may average 200 lb. At the base (Elevation 350), the river divides into two channels, the Rio Hondo, flowing southwest, and the East San Gabriel, flowing south. At present, by far the largest portion of a flood finds its way into the Rio Hondo.

On the débris cone much seepage occurs into the underlying gravel beds, particularly during heavy storms. The percentage of seepage increases as the débris becomes smaller and more uniform in size. The writer's experience has shown that an aggregate, such as is generally used for concreting, presents an excellent seepage bed, while large boulders and coarse gravel reduce the percentage of absorbing voids. Still better results are obtained with fine sands, but they are not always to be found on the cone. The water-plane under a débris cone is, as a rule, at considerable depth, from 100 to 200 ft. or more below the surface; so that storage basins of large capacity are often available.

During the peak of floods the quantity of silt in suspension is large, and such muddy water is not absorbed. It has become the practice in artificial spreading works not to divert any water during the

first day or two of a flood, so as to eliminate the muddy water which would silt up the pores of the gravel beds. If, therefore, artificial absorption is practiced to reduce a peak load, the first requisite will be a material reduction in the quantity of silt in suspension. The greatest difficulty so far encountered with unimproved streams, however, is the instability of the channel on the cone and the danger of causing diversions, for which the manipulator would be responsible.

The action of a heavy flood in most channels is to tear up the stream bed and produce temporary scouring. Subsequently, during the receding of the river, the scoured bed is refilled. Since streams always carry some silt, the effect of a receding stream is to silt up the pores in the gravel of a débris cone, making it almost water-tight. A great deal depends on the formation which the stream traverses in its upper course. Wherever there are lime deposits or clay, the sealing process is almost perfect. The best example of such silting is probably the Whitewater River near Banning, which most of the time has a milky appearance from the silt it carries, which silt is extremely effective in sealing the pores after the winter floods.

It is essential, if the natural seepage is to be maintained, that floods should occur, which will tear up the river-bed and re-open the pores. The tearing-up process is the most important function of a flood and furthers natural absorption.

The changes in the course of a stream on its débris cone presents a criterion from which conclusions might be drawn as to the quantity of débris produced in the mountain section. No information has come to the writer's knowledge that either the San Gabriel River or the Santa Ana River has materially changed its course during the last 60 years.

The improvement contemplated by the Los Angeles County Flood Control District for the San Gabriel River cone is described in its Report of 1917, page 29, as follows:

"The work required is the excavation of a channel 400 ft. wide and 4 ft. deep to form a minor or interior channel. The material from this excavated channel is to be placed in levees upon either side of the channel, and at a distance varying from 600 ft. to ½ mile, as shown upon the Plate. This boulder material is to form levees which will be 12 ft. wide on top, the slope on the water side will be 3 horizontal to 1 vertical, and upon the back or land side will be 1½

horizontal to 1 vertical; the elevation of the tops of these levees to be 18 ft. above the bottom of the minor or interior channel. This minor channel will provide for the carrying of ordinary floods, say, up to 20 000 sec-ft., while when the great floods of 40 000 or 50 000 cu. ft. per sec. come, the entire area between levees amounting to 2 500 acres will be flooded, thereby providing a large spreading ground. This design of a minor and major channel will have a tendency to hold the principal thread of the main current near the center of the minor or deeper channel, and thereby reduce the velocities near the levees, on the whole having a tendency to arrest velocity and precipitate the detritus, and to cause as much of the flow as possible to percolate and reach the artesian water through the coarse materials at the head of the valley. * * *

"* * * It should now be distinctly understood that the plan of the channel here outlined, and, for that matter, any other channel in a detrital country, will require examination and correction after every flood. Great floods coming out of the San Gabriel canyon always, when at peak, bring great loads of detrital matter which are dropped by a very sensitive natural law as the velocity or carrying power of the stream weakens with every slight reduction of the gradient, and a heavy part may be dropped by one storm a short distance down this very large channel, while the next flood wave being larger or smaller, may drop its load farther down, or higher up. This condition must be watched with a vigilant eye and removed so thoroughly after each storm as to bring both the minor or interior channel and the larger or major channel to a carrying capacity. If this maintenance is neglected, disaster will follow."

Effective control of floods on the débris cone requires, first, that the stream arrive at the mouth of the canyon delivered of its rolling débris; second, that the stream be prevented from resuming erosive action on its way down the cone; third, that the stream arrive at the mouth of the canyon delivered of the larger portion of its sediments in suspension; and fourth, that means be provided to induce the seepage into the ground of a large percentage of the flood flow, particularly of the peak load.

The Valley Section.—A stream, having delivered itself on the débris cone of its burden of coarse detritus, such as boulders, large gravel, and coarse sand, has a scouring capacity from the point where it leaves the cone, and, therefore, will produce a stream-bed in cut. Proceeding down stream the cut gradually becomes shallower and a

deposit of sediment takes place. With most rivers there occurs in the valley section a succession of erosional and alluvial stream activity.

A channel in cut has a more or less rectangular cross-section, sometimes with well-developed berms. The formation in the valley districts is an alluvium, consisting of gravelly or sandy loam, which becomes finer with lower elevations, constituting river banks that readily succumb to scouring action under high velocities. The floor of a channel consists of different materials than the banks. The high flood velocities of a cutting stream have a distinct scouring effect on the bottom, and the materials forming it are, during the flood flow, in motion to great depth. A large quantity of sand and silt, therefore, is liberated and set in motion, which ultimately will be laid down where grades become flatter and velocities lower. This débris in traction undergoes a natural process of assortment, the heavier sinking while the lighter remains near the top, with the silt floating off in suspension. A test hole in a river-bed would show a fairly well assorted deposit, ranging from sand on the surface to gravel and small boulders at greater depth. As the flood recedes the bed is refilled, and over a series of years its profile and depth remain about the same.

From observations on the Santa Ana River by Mr. G. O. Newman in 1892, it was concluded that the deposits forming the bed at the point of observation were in motion during flood to a depth of at least 15 ft.

The volume of débris liberated and moved on account of erosion of the channel floor is many times greater than that produced by washing of banks. Scour of the channel floor is at its height during the peak flow, while undermining of banks is, as a rule, more pronounced during a protracted receding flood, as previously mentioned. The damage in an erosional section is caused by undermining of the banks, not by overflow.

The effect of the erosional action and the loading of a river becomes manifest proceeding down stream under a flattening gradient. The channel becomes gradually shallower and the stream drops its débris and eventually overflows its banks. This process is sometimes so pronounced as to produce a secondary débris cone with its characteristic steepest gradients along lines running through its apex. An example is to be found in the Los Angeles River which, some years ago, had its course from Fruitland south, passing between Bell

Station and Huntington Park, while to-day it flows from the apex of its secondary débris cone at Fruitland in a southeast direction, passing east of Bell Station.

As a rule there is a pronounced small channel in these alluvial sectors.

Fig. 5 is a cross-section of the Rio Hondo at the Stewart and Gray Road, just below Downey, where the river begins to overflow. The

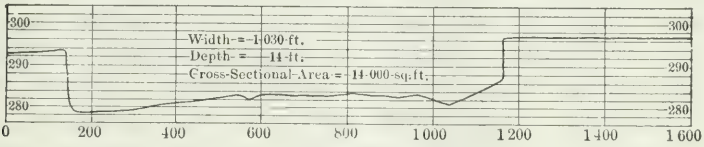


FIG. 3.

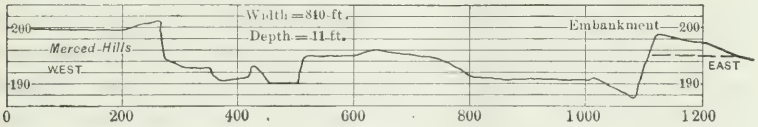


FIG. 4.

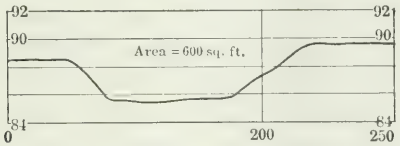


FIG. 5.

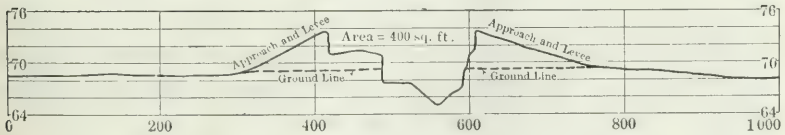


FIG. 6.

cross-sectional area is 600 sq. ft. as compared with 14 000 sq. ft. at Pomona Boulevard, as shown on Fig. 3. (See also Fig. 6.)

Below an alluvial section there invariably follows an erosional one, and *vice versa,* until the stream is absorbed by a larger one or enters its delta. For the Rio Hondo the last erosional sector is located between Downey and Dominguez, below the junction with the Los Angeles River. The slower velocities in an alluvial or inundated section are favorable for the absorption of flood-water, provided the

water is reasonably clear and the water-plane at some depth below the surface.

It would appear rational to assume that after a long period of years a stream would change its activity in a locality from alluvial to erosional, and *vice versa.*

There is a wide difference in the character and volume of flood flow from forested or brush-covered mountainous water-sheds, as compared with urban or interurban drainage areas. Drainage channels issuing from the latter have no débris cone and move a large quantity of detrital matter into the valley section. The effect of this is noticeable particularly during a series of years of moderate or deficient rainfall. Years may pass without the heavy rains which produce a moderate flood run-off in the mountains, while occasional short but intense showers may cause a flashy run-off from city and interurban streets, of sufficient force to carry a considerable load of sediment into the smaller natural drainage channels. The volume of flood flow may be sufficient to reach a main drainage artery; being without support from the mountains, it may not reach tide-water, but slowly seeps away. It also occurs that a flood flow may be of such short duration that it will not pass beyond a minor branch channel.

However, in each case, there will be a deposit of detrital matter, at least along portions of secondary channels, and sometimes over portions of a main channel. The cumulative effect over a period of years of such stream activity is a partial filling up of the affected river-beds, with the result that they are not in shape to take care of a sudden exceptional flood. The remedy commonly resorted to is to provide a compound cross-section with a small central channel and wide berms. In order to be effective, however, this type of construction must be carried clear to the sea.

The present methods of improvement in the valley section are bank protection by pile and wire entanglement for erosional river-beds, and levees for alluvial ones. No attempt has been made with larger streams to check bottom erosion by the introduction of fixed sills.

A peculiar feature of the valley zones are the "narrows" which form what might be called fixed points in the valley course of a river, where the river must pass regardless of what action might take place in its course above or below. The effect of narrows is mani-

fested not only on the surface streams, but also on the underflow, in that the contraction of the valley brings the underflow to the surface, and produces artesian or rising streams and artesian flow in cased wells.

Reviewing the characteristics of the valley section, it is found that the flood channels appear as alternation of erosional channels and more or less pronounced alluvial cones. The stream reaches the valley section with a capacity to increase its burden, and invariably leaves it with a full load. Damage is caused by the washing of banks in erosional channels, and by inundation in alluvial ones.

River control in the valley section requires the prevention of erosion and the transportation of detrital matter, alternating with protection from inundation, with the ultimate purpose of delivering at the delta a stream free from débris in traction, and clear of the major portion of its sediments in suspension. Improvements on the débris cone and in the mountain section above, unless resulting in a reduction of volume of stream flow, are of no material beneficial effect in the valley section. In fact, an attempt to clarify the stream and to deliver clear water into the valley section would be positively injurious, unless accompanied by the introduction of flatter gradients.

Exceptional as well as small flood discharge must be taken into consideration, and particularly the cumulative effect of exclusively moderate or small flood flow over a series of years.

The Delta Section.—In the delta section, grades continue to flatten down to such an extent that the action of the river becomes alluvial, building up a cone of sand and silt. As a rule, the stream channel is quite shallow and of small carrying capacity, so that even ordinary floods produce inundations. Shifting of the channel occurs more frequently than on the débris cone. A single flood may produce deposits over large areas, of 1 ft. or more, as has occurred near Long Beach during the floods of the Rio Hondo-Los Angeles River.

For this river the delta starts at Dominguez at an elevation of 50 ft., about 10 miles from tide-water, with a grade of 7 ft. to the mile, while in the vicinity of tide-water it falls to 2 ft. per mile. Fig. 7 gives a cross-section of the delta between the Dominguez and Long Beach Mesas. It will be noticed that the country is flat and that there is no pronounced channel of large capacity. This area has been inundated during heavy floods over its entire width.

If natural conditions, such as grades, volume of flow, and silt, would permit a solution of the problem in the delta district, stretches would be found where the stream-bed is in a state of stable equilibrium, and such stretches would indicate the grades and character of channel required. In a typical delta, the writer has nowhere found such conditions.

At the estuaries of streams are found almost invariably large tidal flats, often with a considerable number of tidal channels and lagoons. The banks of the tide flats are about at the elevation of high tide, between 5 and 6 ft. above mean low water. Where the area occupied by the tidal channels and lagoons is of considerable extent, so as to produce a tidal prism of some magnitude, the estuary is open all the year round, regardless of the fact that the rivers dry up in summer, as, for example, at Alamitos Bay, Newport Beach, Playa del Rey, and Long Beach. In many streams, however, the tidal prism is not sufficiently large, and breaker action closes the estuary soon after

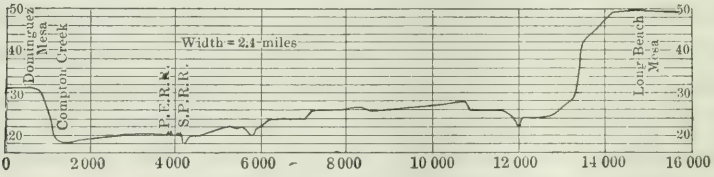


FIG. 7.

the winter floods, to remain in this condition until the next flood. These conditions are observed at Santa Barbara, at Hueneme, and in many estuaries of the minor rivers.

The average rise of tide in Southern California is between 5 and 6 ft., the maximum being between 8 and 9 ft.

There is a strong littoral current along the coast of Southern California, running in a southeast direction. Its effect has been to shift the mouths of the streams down coast, there being a constant scouring action on the south side of the estuary. This has come to the writer's notice particularly at Long Beach. Some of the older coast survey maps which show the estuary at Long Beach, show the outlet about ¼ mile west of the present lift-bridge of the Salt Lake Railroad. When in the early Nineties, the Los Angeles Terminal Railway, now the Salt Lake Railroad, built its trestle across the estuary, it was found difficult to prevent the scouring of the eastern

embankment, and many carloads of boulders and scrap iron were dumped to consolidate the east bank at the crossing. The bridge was about 500 ft. inland from the high tide line. However, the littoral current was strong enough to produce a deflection along the beach, as shown in Fig. 8.

The deflection of an estuary continues until checked by a resisting formation, like the shale hills of Playa del Rey, or the clay mesas of Seal Beach, Huntington Beach, and Manhattan Beach.

If it happens that a flood is great enough to cut through the sand spit at its land end, a direct outlet into the ocean will be established

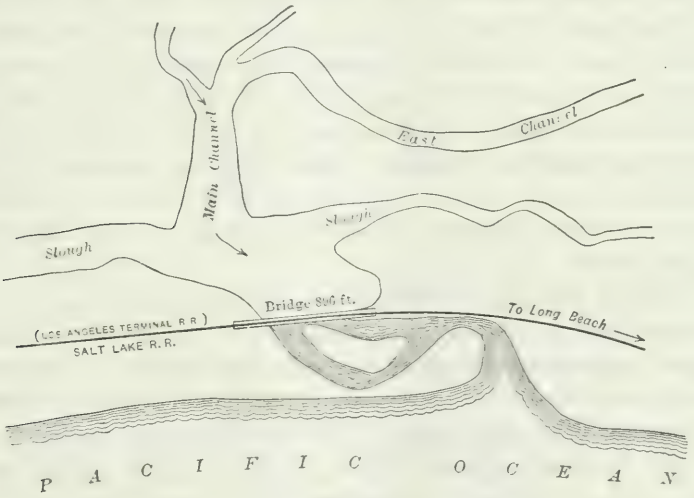


FIG. 8.

along the lines of steepest grade, as occurred with the Santa Ana River during the flood of 1916, when it abandoned Newport Beach lagoon and estuary, and cut a direct channel to the beach about 2 miles west of the estuary.

After such a direct outlet is cut, the littoral current immediately resumes its action tending to shift the estuary again to the south. Eventually, the effect will be a gradual obliteration of the old sand-spit and the formation of a new one in the wake of the shifting estuary.

An example of a river the delta of which begins only at tide-water, is the Santa Clara in Ventura County. The peculiarity of this river is

that from Santa Paula to tide-water, or a distance of 16 miles, the grade changes very slightly, namely, from 16.7 ft. to 13.5 ft. per mile. As a consequence, the river is in cut practically to the beach. At the State Highway bridge near El Rio, about half way between Ventura and Oxnard, and 4½ miles from tide-water, the river runs still in a well pronounced cut, at an elevation of 62 ft. above sea level. Charles W. Petit, Assoc. M. Am. Soc. C. E., County Engineer of Ventura County, has stated that the elevation of the river bottom at the bridge did not change at all between 1896 and 1912. The problem on the lower Santa Clara River is, therefore, clearly one of bank protection.

Summarizing conditions in the delta section, it may be stated that they are those of a river running on a flattening grade, and therefore unable to carry its load of sediment. The result is the building up of a delta, or sedimental cone, over which the stream spreads and shifts its course with every large flood. It is not possible to improve the general condition of grades. Some relief may be gained by concentrating the flow between levees and giving the channel the shortest route possible to tide-water, as is proposed for the Rio Hondo-Los Angeles River.

However, the basic trouble is the sediment, and this can only be eliminated by preventing its accumulation in the preceding erosional section. A stream arriving at the head of its delta in a more or less clarified state, would have a capacity to erode and cut a deeper channel, even on the flattening gradient of the delta formation, and it is conceivable that during moderate floods it might be able to take care of itself without further improvement. For extraordinary floods, the inundation area should be defined by levees and its use for residential or industrial purposes prevented.

The littoral current which, in this section of the coast, tends to deflect an estuary in a southeast direction, will exercise a marked influence on the land water flood problems.

DISCUSSION

CHARLES A. POHL,* M. AM. SOC. C. E.—Attention is called, on page 1128 of the paper, to observations by Mr. G. O. Newman, in 1892, on the Santa Ana River, which show that deposits in the river bed were in motion to a depth of 15 ft. Mr. Pohl.

Observations on other rivers show that scour of the river bed may extend to even greater depths. In a report by the late James B. Eads, F. Am. Soc. C. E., on the St. Louis Bridge, he states:

"Soundings made by me prove that this deposit is scoured out to a great depth in time of floods and freshets. Although I have not had any extreme stage of water in which to make my observations, I found that a rise 13 ft. less than high-water mark caused a scour of 18 ft. The greatest variation in the height of the river known at this place is about 41 ft. * * * The concentration into this narrow channel of the vast volumes that are sometimes poured out of the gigantic net-work of streams above St. Louis, the main artery alone of which is navigable over a thousand leagues above this city, assures me that in time of floods it is not improbable that this deposit is removed to twice or thrice the depth shown by my soundings, and perhaps to the rock itself."

The rock under the westerly pier of the Eads Bridge is 55 ft. below high water; under the second pier, it is 99 ft., under the third, 127 ft., and under the fourth pier, at the easterly side of the channel, 135 ft. below high water. At the center of the middle span, the low-water depth of the river was about 15 ft., and the depth from the river bed at the same point to rock at low-water stage was 60 ft. Under the center of the easterly span, the low-water depth was 15 ft. and the depth from the river bed to rock about 80 ft. As Capt. Eads' observations were made presumably at the bridge site, they would indicate a scour of four and possibly five times the low-water channel depth.

The extent of the erosion of the banks and bed of the Mississippi River is enormous. The following data are taken from a Report by a Special Board of Engineers on Survey of Mississippi River:†

"Similar measurements were made during the present year (1908-09) as to the extent of caving banks on the river between Cairo and St. Louis, by which it was shown that the volume of earth falling into the river from that reach averaged annually 64 000 000 cu. yd. from 1879-1889; nearly 48 000 000 cu. yd. from 1889-1899; and about 49 500 000 cu. yd. from 1899-1907, the diminution of such caving in the last eighteen years evidenced thereby being considered due to the extensive revetments that were built along this reach between 1881 and 1905.

* * * * * *

* New York City.

† Document No. 50, 61st Congress, 1st Session.

Mr. Pohl. "AVERAGE ANNUAL AMOUNT OF CAVING ON BOTH BANKS, MISSISSIPPI RIVER, CAIRO TO DONALDSONVILLE.

Locality.	AREA.	HEIGHT FROM RIVER BOTTOM TO TOP OF BANK.
	Square yards.	Feet.
Cairo to Memphis	6 112 000	53
Memphis to Helena	2 978 000	47
Helena to Arkansas City	4 137 000	62
Arkansas City to Vicksburg	12 024 000	67
Vicksburg to Natchez	9 371 000	76
Natchez to Red River	3 106 000	73
Red River to Donaldsonville	1 263 000	84
Total	38 991 000	66*

* Annual average.

* * * * * *

"Experiments conducted in 1879 under the direction of the St. Louis U. S. Engineer Department at the mouth of the Missouri River to determine the amount of sediment coming into the Mississippi River from the Missouri above St. Louis (p. 1653, Annual Report, Chief of Engineers, 1881, and p. 3090, Annual Report, Chief of Engineers, 1887), showed that the floating sediment alone, resulting from caving banks in the Missouri itself, amounted to 187 000 000 cu. yd. during nine months of the year. As these observations took no account of the heavier matter in the foot of depth nearest the bottom, nor of the solid material (such as sand or gravel) driven along the actual bed of the river by its currents, Col. Suter considered it safe to assume that fully 400 000 000 cu. yd. of material then came out of the Missouri River in twelve months. It is probable that at least the same amount of material is still coming out of the Missouri River at the present time. Other observations by the U. S. Engineer Department and the Mississippi River Commission indicate an outflow of sediment and rolling material of about 36 000 000 cu. yd. per year from the Ohio River, about 5 000 000 cu. yd. per year from the Arkansas River and about 6 000 000 cu. yd. per year from the Red River.

"By the above showing, the amount of material originating from caving banks which the Mississippi River, between St. Louis and permanently deep water in the river below Donaldsonville, must carry along and dispose of every year, amounts to more than one and one-fourth billion cubic yards."

One of the factors in the adverse report by a Special Board of Engineers on the improvement of the Mississippi River between St. Louis and Cairo, by means of two large dams—one at Jefferson Barracks just below St. Louis, and the other near the Thebes Bridge, about 40 miles above Cairo—was that the filling up of the pools above the dams would inevitably take place.

An estimate was made by the Board that dredging of from 100 000 000 to 400 000 000 cu. yd. of material would be required each year to prevent the filling up of the channel above the dams and the floods and other damage resulting therefrom. Mr. Pohl.

The Tennessee River does not carry a heavy load of silt. Its banks are very stable, and very little caving and erosion occur along the main stream which has a length of about 215 miles above the Hales Bar Dam, near Chattanooga, to the head of navigation at Knoxville, and a drainage area of about 29 000 sq. miles at the dam. The material carried is nearly all in suspension and presumably comes from the smaller tributaries 100 miles or more.

In 1918, fields of corn were under cultivation above this dam on land which was under low water when the dam was completed in 1913.

The raising of the river bed by several feet in 5 years shows what may be expected where the velocity of a river carrying soil in suspension is lessened by the construction of a large dam.

FRANCIS H. WRIGHT,* ASSOC. M. AM. SOC. C. E.—The losses caused by erosion along the Mississippi River are something which erosion has also built up again, that is, the eroded material becomes alluvial as it is restored by deposit along the lower reaches. The speaker does not believe that, if the quantity of eroded material is set against the quantity restored along the lower river, any great change will be found in the adjacent geology within the span of a life time. He believes that the great quantity that is carried out of the mouth of the Mississippi is that which originally came from the upper reaches of the Missouri River, or from the Ohio River, especially the former. Mr. Wright.

The gradient of a stream near its source in the hills begins as a convex curve which gradually changes and becomes concave, ending in a tangent at its base or at sea level. The convex gradient is one of irregular steep slopes making rapid velocity, with great scouring and conveying capacity. The concave gradient of more uniform slopes is characterized by their flattening with approach to base level, with reduction in the average velocity, and tendency of the stream to deposit its burden. This tendency is apparent immediately following periods of flood.

The section of a stream within and below the elevation of what is commonly known as "bank full" stage can be defined as a normal section. Equal velocities through equal sections will convey equal volumes in equal periods of time. Variations in velocity, however, do not function with variations that exist in the normal section and the volume of discharge of a stream. The normal section along the concave gradient becomes abnormal during flood, through the overflow, and the previously mentioned variables are thereby harmonized.

* New York City.

Mr. Wright.

Unless confined, the section becomes greatly increased, with a consequent reduction in the average velocity and carrying capacity. Subsidence brings a further reduction in velocity, and the stream returns to its normal section within its banks. These reductions in velocity at a time when the stream is most charged with suspended matter, show the greatest amount of deposit. This is especially apparent over the overflowed area just after subsidence. The removal of material from along the upper reaches of a stream to its deposit along the lower reaches is a natural result from the character and variation in its gradient. The Mississippi being very flat and alluvial in its delta region, it follows that its gradient at that point is a concave curve.

The condition described in Mr. Sonderegger's paper, as the speaker has been able to understand it, is a more difficult problem of flood control than that of the Mississippi, even though of lesser magnitude.

These Southern California rivers pass suddenly from steep slopes in rock-lined canyons through débris cones of unknown density and doubtful stability to an alluvial valley with flat slopes. These cones, accumulated through ages, are located where the stream section readily becomes abnormal and where the velocity suddenly decreases.

Among the requirements which the author considers as the solution of this problem is that of absorption on the débris cone of the peak flood load. This implies the use of storage reservoirs. The speaker can imagine that it would be quite difficult to hold this water on these cones. It appears possible for it to escape, not only through absorption, but by breaking through the cone at some lower elevation and seeking a new channel to the sea. If the escape is only through absorption within the débris cone, the seepage arising therefrom in the lower valley would have to be considered.

Some of the effects of high water which the speaker has observed along the Lower Mississippi have led him to believe that full storage reservoirs would cause all the adjacent soil to become saturated. A continued rainy period, added to a full reservoir, would furnish more potential danger than without the reservoirs.

With reference to the previous remarks about the erosion along the Lower Mississippi, the speaker does not wish to convey the impression that he considers it as insignificant. It is very great, but it is balanced by the replacement which is also very great. During the past ages, the latter has exceeded the former, or there would be no alluvial valley to-day. The sum total of material that passes the mouth of the Mississippi River into the Gulf of Mexico will balance the quantity derived from the region tributary to the convex gradient of that river or its feeders, less the increment in the lower alluvial valley.

Mr. Wright.

As to actual erosion, one instance is recalled where a plantation (the Bondurant Plantation in Tensas Parish), in Louisiana lost nearly 1 700 ft. of receding caving bank during one year. This distance was taken along a line at right angles to the axis of the channel at that place and was measured from actual surveys. A few miles down stream, a sand-bar was built up, which was probably of sufficient volume to offset that which was taken from this plantation. That which is taken from one side is restored on the opposite side, etc. The silt is deposited, picked up, carried farther, again deposited, etc., during the different years of its journey from its origin to its final deposit or to the sea.

J. A. Ockerson, Past-President, Am. Soc. C. E., has made a report* concerning surveys undertaken to estimate the amount of erosion through caving banks along the Mississippi River. These surveys were made in 1891-92 along caving bends between Cairo, Ill., and Donaldsonville, La. The survey of 1879 of the Mississippi River Commission between Cairo and the Passes was a thorough topographical and hydrographical survey which included the areas resurveyed, and an accurate estimate was made from the sections produced between the two surveys.

The estimate given in that report shows that the annual erosion between Cairo and Donaldsonville amounts to a volume 66 ft. in depth, over an area of 9½ acres for each mile of river, or, for the total length of river included by the survey, a volume of 86 ft. in depth over 10 sq. miles of area. This estimate only refers to the quantity eroded and makes no allowance for quantities deposited on banks or in the channel.

Mr. Ockerson mentions a fill near Commerce, Mo., about 2 000 ft. wide and about 5 miles long, which was under cultivation at that time (1891), but which, in 1879, was occupied by the river and outside of a caving bank. The large quantity of erosion referred to in Mr. Ockerson's report might have originally started in Wyoming or Montana and probably required some hundred years to reach the Lower Mississippi, perhaps less, but a long time.

It is said that, in the distant past, the mouth of the Mississippi River was at about the present location of Cairo, Ill., and that there it entered the Gulf of Mexico. The region to the south, which is commonly known as the delta, has been built up through alluvial deposits since that time.

The construction of a concrete warehouse for an unloading wharf on the Mississippi River, on which the speaker was engaged, required pile clusters, heavily loaded, to support the concrete footings. A sheet-piling enclosure surrounded the site, and after the clusters were driven, a sand filling was pumped in, surrounding the clusters to the

* Annual Report of the Chief of Engineers, U. S. A., 1892, Pt. 4, p. 3111.

Mr. Wright. elevation of the bottom of the concrete footings, and serving as a base for them during construction. At extreme low water, or while the sump pumps were active, a crust formed over this fill, which was able to carry a heavy load and on which one could walk. A 70-ft. pile dropping from the driver leads would not "take up" for 30 or 40 ft. in the silt below. Piling in place and projecting 5 or 6 ft. above the crust vibrated very perceptibly from a good lusty kick. This sand filling had been placed during an interval of perhaps two years from the placing of these piling clusters until it was practicable to pour the concrete footings.

In checking the positions of the piles in the clusters for errors in their centers of gravity and their correction, the speaker observed that many of them were much inclined from the perpendicular and symmetrical in the amount of their inclination and in its general direction. It was surprising to discover that the pitch from the perpendicular of some of the piling was 3 in. and more to the foot of length, or a lean in a 70-ft. pile of about 18 ft. It was realized that the design load could not be applied under these conditions, as the piling would bend through the soft silt to complete failure. A careful survey was made with a plumb and rule, and those piles which deviated beyond the adopted limit were cut out and others driven. The symmetrical inclination was attributed to a movement of the sand filling toward the channel of the river.

It is the speaker's opinion that movement of a culvert is possible through inclined piling in a soft soil. Inclined piling has been driven and properly capped to receive thrusts without bending in the piles and has been satisfactory. Where compensation against bending is not provided, or cannot be given, the consequences are apt to be uncomfortable to say the least.

Mr. Dunham. H. F. DUNHAM,* M. AM. SOC. C. E.—The profile of the Susquehanna River could be cited as being unlike any of the profiles referred to by the author. The grades coincide or are approximately regular until the flattening out would indicate a near approach to sea level, and then the river drops more than 200 ft. in about 40 miles, or a comparatively short distance. This occurs in a region where uplifts and vibrations are rare instead of frequent. It has not been shown that the condition is due to glacial action.

Two or three statements in this interesting paper may seem worthy of further comment or of explanation by the author. His graphic description of débris cones is appreciated by the speaker who has had little opportunity to see them and none at all to study them. In ignorance, too, of California practice, any detail of the method used to determine the percentage of seepage at the higher or lower

* New York City.

Mr. Dunham.

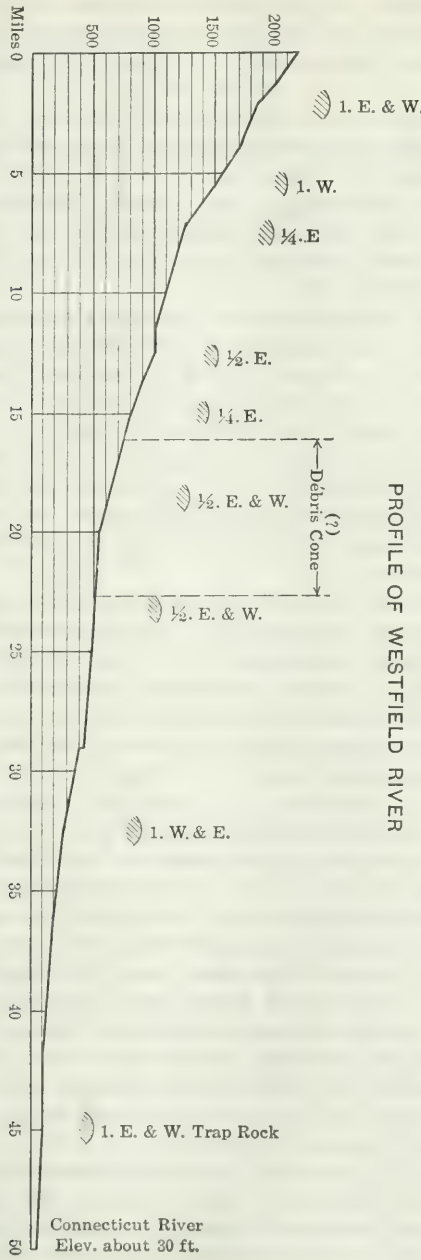


FIG. 9.

Mr. Dunham.

sections of a débris cone would be of interest to him, especially as the distance between the points of variation are well within ten miles, and the formation throughout is the result of moderately uniform natural causes.

Except for some subtle condition not understood, the speaker would say that the more an aggregate differed from one that ought to be used for concrete, the better seepage bed it would be where large additions to the ground-water were desired. The weight of the material composing the aggregate might be given in terms of specific gravity of the rock formations from which it was derived, whether in part volcanic ash or tufa.

On page 1127, requirements for the effective control of floods on the débris cone are mentioned. They comprise elimination of the rolling débris and the larger part of the suspended matter when the water reaches the mouth of the canyon; and it is also required that flood water so deprived of its burdens shall not take up new material or erode its bed and banks on a grade of 50 ft. to the mile. What agency can make this possible? Where on the steep slopes and mountain sides are reservoirs to be provided above the canyon for flood control? If provided, how quickly would they be filled with débris and become useless?

A section of the profile of the Westfield River, in Massachusetts, compares fairly well for grade with the débris-cone channel section (Fig. 2), shown by the author. The area above comprises about 90 sq. miles. Rocks, granite, gneiss, and mica schist are present; and a small quantity of slate is also present. The precipitation measures about 40 in. per annum, more nearly uniform per month, by the average of many years, than can be found in many sections of the United States. Downpours of 3 or 4 in. in 24 hours have been observed, and, in addition, there are heavy snows and thick ice, in the annual freshet period. At times of such floods, the depth of the water ranges from 15 to 20 ft., or more, above low water. The river flows over glacial deposits of boulder clay in many places. The river bed is never quite dry, but extremely few areas are covered with sand. At rare intervals there have been days when no current was visible. It is a river bed of stones, with boulders up to many tons in weight, and rather uniformly distributed, that is, big boulders are not appreciably more common or numerous at the upper than at the down-stream end of the cone section. The ice is a carrier, as it is often frozen to the boulders and stones, and more often bowls them along. At sharp turns in the channel, there may be depths of water of from 5 to 15 ft., measured from the low-water mark, and, in such holes, the scour at flood and the re-filling would be noticeable, but all within a very few feet; it might be nearer the truth to say a very few inches. Over or on the surface of the river bed, there is a slow

Mr. Dunham.

average movement of the stones and boulders down stream, but a movement of such material to a depth of 2 ft., or even 1 ft., below low-water mark, would be contrary to all observations unless it could be shown that at flood time the material at a depth of "15" ft. was moving down stream, while the river bed remained in place. If given in detail the observations of Mr. Newman, mentioned under "The Valley Section" (with a grade of 15 ft. per mile), should show some marked difference in the character of the material in California and New England, if the grades and moving agents were nearly identical.

With a rapidly increasing flow, water would find its way into the material of the river bed and banks or shores, and doubtless would exert pressures tending to keep that material in place. The pressure would be reversed when a flood-plain was quickly lowered, but at any time only a slight down-stream movement of the river bed can be expected and that over only limited areas.

A profile of the Westfield River (Fig. 9), shows a section which may correspond to that of the débris cone. A few adjacent summit elevations with distances from the river channel, in miles or fractions of miles, are shown. Thus, "$\frac{1}{2}$ E" shows the elevation of the ground $\frac{1}{2}$ mile east of the river. The elevations are taken chiefly from U. S. Geological Survey quadrangles.

Mr. Bowlus.

FRED D. BOWLUS,* ASSOC. M. AM. SOC. C. E. (by letter).—The author mentions two important factors that put the flood-control problems of Southern California in a class by themselves; first, the uneven distribution of the rainfall, and second, the short distance from the mountain heights to the sea, with consequent steep gradients. Both contribute toward causing the erosion and transportation of enormous quantities of materials toward the ocean, which materials range from the finest silts to huge boulders.

A comprehensive study of the conditions affecting the transportation of these materials, as well as of the hydraulic properties of the existing channels, is of the utmost importance in the solution of flood-control problems. Without knowing these conditions, it is difficult to design a channel to carry a certain quantity of water, when the channel bed is of sand or gravel. During a flood, the bottom is scoured out, so that the channel carries very much more water than it is designed for, and as the flood recedes the channel bed is refilled with material. The depth of scour and of refill is difficult to determine.

Fig. 10 has been prepared to show the relative elevations of the water surface and of the average stream bed of the Colorado River at Yuma, Ariz., for four months of 1914. Generally, the floods occur on the Lower Colorado in June or early in July. The year 1914 represents an average of the floods for the past few years, the crest discharge being 137 000 sec-ft. Fig. 10 shows the gauge heights and

* San Francisco, Cal.

Mr. Bowlus.

mean depths. The top line represents the water surface, and the lower line the mean elevation of the stream bed. The discharge of the river is also indicated. It will be noted that the bed was lowered more than the water surface was raised during the period of the flood. The channel at the Yuma Station is somewhat confined, being between 500 and 600 ft. in width, and is thus illustrative of the conditions that would prevail with levees on flood-controlled streams. The drainage area of the Colorado River is about a thousand times greater than that of the San Gabriel, while the peak discharge is only four or five times larger. The duration of the flood, however, is much longer on the Colorado River. A study of the diagram shows that the erosion of the bed materials is gradual. More than 20 ft. in depth were eroded during this particular flood, all of which was replaced later. This represents the moving of about 1 000 000 cu. ft. of material to each 100-ft. station along the river bed. The extreme range

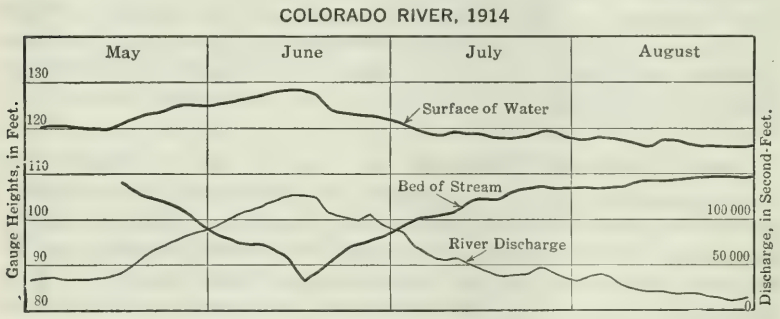


FIG. 10.

in the elevation of the river bed at Yuma is 40 ft. The mean velocity, at flood crest, never exceeds 7 or 8 ft. per sec., while, according to Mr. Ebert, of the U. S. Geological Survey, there have been peak velocities at the mouth of the San Gabriel River of from 20 to 25 ft. per sec.

Transportation of Débris.—The study of the transportation of débris is very complicated, because of the relationship between the various factors. Some valuable studies have been made by various hydraulic, sanitary, chemical, and mining engineers. Most of them, however, were for specific purposes, and their conclusions cannot be made of general application. The tests of Dr. G. K. Gilbert,* referred to by Mr. Sonderegger, were undertaken for a much broader purpose, but it is regretted that they were not completed as originally planned. All the sand used in Dr. Gilbert's tests was of a size greater than 50 mesh, consequently, conclusions are not directly applicable to the material that is found, for instance, in the Colorado River canals, all of which is finer than 80 mesh.

* *Professional Paper No. 86*, U. S. Geological Survey.

Mr. Bowlus.

It is generally recognized that there are three modes of transportation of silt. The finest particles are suspended throughout the entire cross-section somewhat like a solution, the medium grades are carried along just above the bottom of the stream by upward currents deflected from the bed, and the largest particles either slide, roll, or skip along the bottom.

No discussion of the many theories advanced for the cause of transportation by the first two methods will be undertaken at this time, the discussion being confined to the last named. In the course of engineering investigations with Mr. C. E. Tait, in the U. S. Department of Agriculture, in co-operation with the California State Department of Engineering, a study of the silt conditions along the Colorado River and the irrigation canals taking therefrom has been made. The Colorado River has been sampled when in flood, and very little material coarser than 100 mesh has been found except near the bed. Along the coastal streams of Southern California, gravel and boulders are transported from the mountains several miles out on to the débris cone.

Carrying Capacity of a Stream.—The transportation of débris by a stream is a tax on its energy. The stream can carry more fine than coarse material. Its carrying capacity has to do with the quantity of load and its weight per unit of water. Experiments have been made to determine the relation between capacity and mean velocity. Deacon* used a glass trough and found that the weight varied, approximately, as the fifth power of the mean velocity. Blue† used a small metal trough and found the variation to be about as the sixth power. It is obvious that these smooth troughs do not represent the conditions found in stream beds composed of sand and gravel.

Many determinations of the silt load per foot-width in canals along the Colorado River have been made by the California State Department of Engineering by taking samples of water at many different points in the cross-section of a canal or stream. That the load increases with the mean velocity is clear, but the exact relation has not been definitely fixed.

Size of Material Carried.—The mass, weight, or size of any individual particle that can just be moved by any given mean velocity (or preferably in terms of bed velocity) differs from, but is often confused with, capacity. It is said, in a loose way, that the quantity of material transported varies with the sixth power of the mean velocity. As a law, this refers to the mass or weight of single particles. It was first suggested by Leslie and Hopkins about 100 years ago, and Wilfred Airy gave proof of it in 1885.‡ This conclusion is based

* *Minutes of Proceedings*, Inst. C. E., Vol. XCVIII, p. 93.
† *Engineering and Mining Journal*, Vol. 84, p. 536.
‡ *Minutes of Proceedings*, Inst. C. E., Vol. LXXXII, p. 25.

Mr. Bowlus.

on the fact that the impulse of a jet of water composed of equal parallel filaments varies as the square of the velocity and that the mass or weight varies as the cube of the diameter, making the weight vary as the sixth power of the velocity. As a matter of fact, on account of the greatly increasing velocity just above the bed of a stream, the particles are moved more easily because of a longer lever arm. Several experimenters have tried to verify this law or to find some other relation.

Dubuat* made some extensive experiments with a wooden trough in the Eighteenth Century, and Blackwell,† of England, made some about 1850. E. H. Hooker‡ has tabulated the results of many of these experiments. Hardly any of the experiments were made with materials resting on those of the same kind and shape. Gilbert, however, made some tests on this basis, and concluded that the diameter of particles transported varies with about the 2.7 power of the velocity, which conclusion is probably based on the mean diameter of the particles in a grade. It would have been more correct to use the diameter of the smallest particles, because those were the first to move, and platting the data on that basis the equation,

$$\text{Diameter} = 0.0032\ \text{Velocity},^{2.6}$$

is found to fit, approximately, indicating that the weight varies as the 7.8 power of the mean velocity.

The foregoing discussion refers, of course, to particles of the same material, specific gravity, and shape. With regard to shape, a round particle will be moved with less velocity than a square angular one. A cylindrical pencil will roll down a very gentle slope, a hexagonally shaped one requires a slope of from 5 to 10°, and one of square cross-section, from 20 to 30 degrees.

No attempt is made here to present the results of the present joint Government and State of California investigation of the Colorado River waters, which are somewhat comprehensive and are not yet fully worked up, but it is hoped that these results may be made available later. It has been the writer's intention merely to suggest the broad scope of this subject and to indicate its very important relation to the flood-control problem, also to call attention to a few of the common errors that are made in discussing the subject and to the confusion that exists with reference to the definition of the terms used.

Although the present investigations have been more or less confined to the Colorado River silt, it is hoped that they will be extended to include other streams of the Southwest, where the conditions are somewhat different.

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* "Traité d'Hydraulique", Paris, 1786.

† *Minutes of Proceedings*, Inst. C. E., Vol. LXXXII, p. 47.

‡ *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 308.

A. L. SONDEREGGER,* M. AM. SOC. C. E. (by letter).—The discussions of Mr. Pohl and Mr. Wright have brought out interesting facts in regard to scouring in times of freshets. Mr. Sonderegger.

Mr. Wright states that losses caused by erosion are "something which erosion has also built up again," and, furthermore, that the great quantities of material carried to the mouth come originally from the upper regions. This is, undoubtedly, true. Bank erosion which, at some points, appears to be enormous and contributes large quantities of silt, is only temporary, and eventually will be obliterated by the action of the river itself and the effect of drift carried by the winds. It also follows, logically, as he states, that on the Lower Mississippi, during past ages, alluvial action has exceeded the erosion. For these reasons, river control in the lower regions by means of levees is only a temporary expedient.

In Southern California, the débris at the mouth of canyons may contain blocks of 1 cu. yd. or more, interspersed with small boulders and coarse gravel. Invariably, it will be found that the blocks gradually disappear within a distance of 2 to 6 miles from the mouth of the canyon, below which point only coarse gravel and sand and some small boulders may be found.

Such steep, short débris cones indicate flashy peaks, with storms of brief duration, and, furthermore, that the quantity of coarse débris, transported in traction in each flood, is comparatively small and that the large blocks are not moved, except during such capital floods as occur only once in generations.

As regards the quantity of silt in suspension, the geological formation of the water-shed and the extent of the brush or timber cover will be of material influence. During the peak of 5 000 sec-ft., of March 2d, 1920, the San Gabriel River carried at Azusa, Cal., 55% of silt by weight, which percentage diminishes rapidly with a falling river. On April 19th, with a flow of 250 sec-ft., the water was clear.

Mr. Dunham has raised the question as to the methods used to determine the relative amount of seepage at higher and lower sections of the débris cone. It is physically impossible to gauge Southern California streams during flood times, at the apex of the débris cone, with a sufficient degree of accuracy to determine seepage losses occurring within a short distance. The beds are too irregular, and the velocities, as high as 30 ft. per sec., produce heavy wave action, so that cross-sections are difficult to determine, while the mass of floating débris makes it almost impossible to operate current meters.

The gaugings shown in Table 1 (page 1148), however, give an idea as to the amount of seepage in the San Gabriel River and other Southern California streams of moderate flow.

* Los Angeles, Cal.

Mr. Sonderegger.

TABLE 1.—GAUGINGS SHOWING SEEPAGE IN SAN GABRIEL RIVER AND OTHER SOUTHERN CALIFORNIA STREAMS.

SAN GABRIEL RIVER.

Date.	Discharge at mouth of canyon, in second-feet.	Discharge at lower point, in second-feet.	Observer.
April 26th, 1903.	285 (Large boulders: slope, 100 ft. per mile.)	229 (At El Monte, 10 miles below mouth; bed, gravel and sand; slope, 28 ft. per mile.)	W. B. Clapp.*
April, 1914.	440	20 (Baldwin Park, 6½ miles below mouth (after a winter with a capital flood in January); small boulders and coarse gravel; slope, 50 ft. per mile.)	Willis S. Jones.
March 31st, 1920.	10.09 (clear) (Eaton Wash; width, 12 ft. E.: cascades: large boulders; slope, 750 ft. per mile.)	224 (Foothill Blvd., 3¼ miles below; width, 6 ft.; small boulders and gravel; slope, 110 ft. per mile.)	A. L. Sonderegger.

SANTA ANITA CREEK.

March 31st 1920.	17.579 (clear) (Width, 19 ft.; cascades; large boulders; slope, 200 ft. per mile.)	11.784 (Foothill Blvd., 1⅝ miles below mouth; width, 14 ft.; small boulders, coarse gravel; slope, 80 ft. per mile.)	A. L. Sonderegger.
" "	17.579	4.679 (Huntington Blvd., 2½ miles below mouth; width, 19½ ft.; coarse gravel; slope, 68 ft. per mile.)	"
" "	17.579	1.800 (Santa Fe Bridge, 2¾ miles below; width, 5 ft.; slope, 57 ft. per mile; sand and coarse gravel.)	"
" "	17.579	(2⅞ miles below mouth; dry; sand and gravel.)	"

* U. S. Geological Survey, *Water Supply Paper No. 100.*

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Paper No. 1450

EXPERIMENTS ON THE FLOW OF WATER THROUGH CONTRACTIONS IN AN OPEN CHANNEL

By E. W. Lane, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. William T. Lyle, D. C. Henny, Karl R. Kennison, R. D. Goodrich, and E. W. Lane.

Synopsis.

In the studies for the design of the flood prevention works of the Miami Conservancy District, the laws governing the flow of water through a contraction in an open channel assumed unusual importance. The available literature on the subject was uncertain and contradictory. To determine the true laws governing the phenomena, the writer carried on, during the summers of 1915 and 1916, a series of small-scale experiments. Although conditions growing out of the war prevented their completion, some interesting results were obtained, and it is believed that they throw considerable light on this little understood subject. The experiments consisted of measurements of flow through:

(*a*) A contraction with rounded edges, 1 ft. wide.
(*b*) A sharp edge contraction, 1 ft. wide.
(*c*) A sharp edge contraction, 2 ft. wide.
(*d*) A short flume with rounded entrance.
(*e*) A short flume with sharp corner entrance.
(*f*) An expanding or Venturi flume.

In the following paper a description of the apparatus and the method of making the experiments is given, together with the results.

Appendixes are attached containing:

I.—A brief description of the application of the theory of flow through contracted openings to the design of the flood prevention works of the Miami Conservancy District.

II.—A description of the unstable state of the stream below a sharp edge contraction.

Briefly stated, the most important conclusions drawn from these experiments are as follows:

The manner in which the flow of water through a contraction takes place, is entirely dependent on the conditions existing in that particular case. The two most common formulas are those of d'Aubuisson and Weisbach. Under certain conditions, one of these formulas may be applicable and, under another set of conditions, the other; while under still other conditions, neither should be used. The paper points out the various conditions of flow, the underlying causes, and the conditions under which they take place. The flow through a contraction is intimately connected with the phenomenon of "maximum flow", the theory of which is well known, but it is herein demonstrated experimentally, and its limitations are shown.

To explain the conditions of flow in the expanding flume, the theory of back-water curves was developed for a frictionless channel with a level bottom and variable width.

Certain forms of contractions were found to possess considerable merit as instruments for the measurement of flowing water.

The Theory of the Flow of Water Through a Contraction in an Open Channel.

The theory of the flow of water through a contraction in an open channel has been an unsettled problem in hydraulics for many years. At the time this investigation was undertaken, although available literature showed many discussions of the subject,* no record of experi-

* "Treatise on Hydraulics", by d'Aubuisson. Bennett's translation (New York, 1857), p. 189.
"Experimental Hydraulik", by Weisbach.
"On the Determination of the Flood Discharge of Rivers and of the Backwater Caused by Contractions", by W. R. Hutton, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 211.
"Treatise on Hydraulics", by Merriman. Eighth Edition, p. 327; Tenth Edition, p. 343a.
"Hydraulics of Rivers, Weirs, and Sluices", by Molitor, p. 32.
"Flow of Water", by Louis Schmeer, p. 180.
"Hydraulic Motors", by Church, p. 227.
"A Report to the Mayor and City Council on Flood Prevention for the City of Columbus, Ohio", by Alvord and Burdick, September, 1913, p. 80.
"The Official Plan of the Miami Conservancy District", Vol. I, pp. 67–71.

ments could be found, which was sufficiently complete to establish any of the theories proposed. Since this study was undertaken, a valuable history and discussion of the various theories has been presented to the Society,* and, therefore, only a brief discussion of the theory will be given in this paper.

Although a number of hydraulicians have proposed formulas to represent the relation of the various factors entering into this problem, they are all based on one or the other of two theories. For convenience, these theories will be given the names of the men who probably first proposed them, that is, the d'Aubuisson and the Weisbach theories.

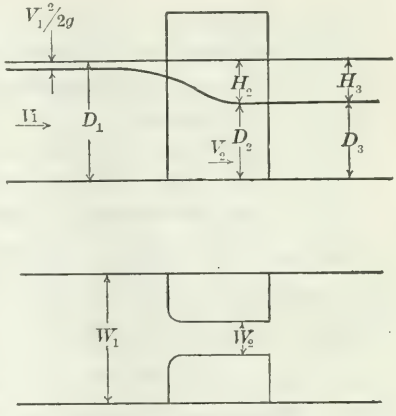


FIG. 1.

Symbols.—Fig. 1 represents the flow of water through a contraction, where:

V_1 = the mean velocity of the water above the contraction.

D_1 = the mean depth of the water above the contraction, including the velocity of the approach head. (The combining into one symbol of the observed depth above the contraction and the velocity of approach head, results in much simpler formulas, the meanings of which are more readily understood.)

W_1 = the mean width of the water above the contraction.

V_2 = the mean velocity of the water in the most contracted section of the channel.

D_2 = the mean depth of the water in the most contracted section of the channel.

W_2 = the mean width of the stream at the most contracted section of the channel.

H_2 = the drop of the water surface at the most contracted section, including the velocity of approach head.

* "Obstruction of Bridge Piers to the Flow of Water", by Floyd A. Nagler, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII, p. 334 (1918).

g = the acceleration due to gravity.

$\frac{V_1^2}{2g}$ = the head due to the velocity of approach of the water above the contraction. This is usually included in D_1.

D_3 = the mean depth of the water in the channel below the contraction.

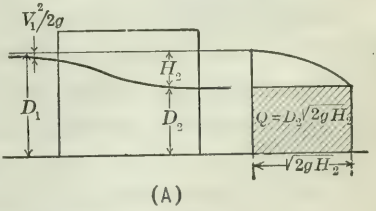
H_3 = the total drop of the water in passing through the contraction, or the observed drop plus the head due to the velocity of approach.

C = a coefficient to take account of losses due to friction, impact, etc. The subscripts, dA and W, designate the d'Aubuisson and the Weisbach coefficients, respectively.

Formulas.—According to the d'Aubuisson theory, V_2, the velocity in the contracted section, is equal to that produced by a drop, H_2, or V_2 is equal to $C_{dA}\sqrt{2\,g\,H_2}$, whence:

$$Q = W_2\,D_2\,V_2 = C_{dA}\,W_2\,D_2\sqrt{2\,g\,H_2} \quad \ldots\ldots\ldots\ldots (1)$$

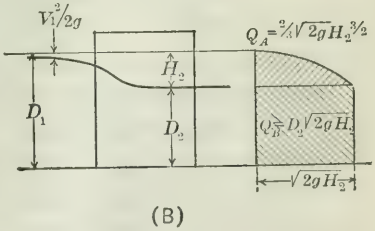
The d'Aubuisson formula is really only an application of the Bernouilli theorem. The discharge according to this formula is represented graphically, for a unit width of channel, by the cross-hatched portion of Fig. 2 (*A*).



(A)

Weisbach considered the flow as being made up of two parts: (*a*) a part equal to that which would flow over a weir with a crest length of W_2 and a head on the crest of H_2, or,

$$Q_A = C_A\,W_2\,\frac{2}{3}\sqrt{2\,g}\,H_2^{\frac{3}{2}} \quad \ldots (2)$$



(B)

FIG. 2.

and (*b*), a part equal to that which would flow through an orifice of width, W_2, and height, D_2, under a head of H_2, or

$$Q_B = C_B\,W_2\,D_2\sqrt{2\,g\,H_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

These two portions are represented graphically, for a unit width, by

the two cross-hatched portions of Fig. 2 (B). In the formula, the two parts are combined with a single coefficient, giving:

$$Q = C_W W_2 \sqrt{2g} \left(\frac{2}{3} H_2^{\frac{3}{2}} + D_2 H_2^{\frac{1}{2}} \right) \ldots\ldots\ldots\ldots (4)$$

It will be noted that Part (b) of the discharge, according to the Weisbach theory, is exactly equal to the total discharge according to the d'Aubuisson theory. The Weisbach theory, therefore, always gives a theoretical discharge greater by the amount of the first part, $W_2 \frac{2}{3} \sqrt{2g} H_2^{\frac{3}{2}}$. In determining the value of the coefficient experimentally, where Q, W_2, D_2, and H_2, are measured, the coefficient, C_W, in the Weisbach formula, for a given experiment, will always be lower than the coefficient, C_{dA}, of the d'Aubuisson theory.

It is usually impractical to determine the depth, D_2, of the water in the most contracted section, on account of the difficulty of measuring the height of the water surface at this point. A common practice is to measure the water surface elevation below the contraction, and assume that the surface in the most contracted section is at the same elevation. For instance, in computing the height of back-water at a bridge opening, the usual practice is to assume that the water surface elevation in the most contracted section is the same as that in the river channel immediately below the bridge. In most cases of bridge openings, this is very nearly the true condition. For some contractions, however, the assumption is far from the truth, and it has a very important effect on the nature of the coefficient which must be used to obtain the true values of discharge. In the computations of the three experimental contractions, this assumption was made, and its effect will be discussed in connection with the results of these experiments.

Description of Experimental Plant.

The experiments made by the writer to study the flow of water through contracted openings, were carried on in a rectangular channel, approximately 6.8 ft. wide and 2 ft. deep. The sides were formed by sheeting which had been used in connection with the laying of a water main, and the bottom was of gravel and silt. The channel paralleled the Mad River and received its water supply from it. The head necessary for the experiments was created by a fall of 2 ft. between the upper end of the channel and the point at which the water discharged back

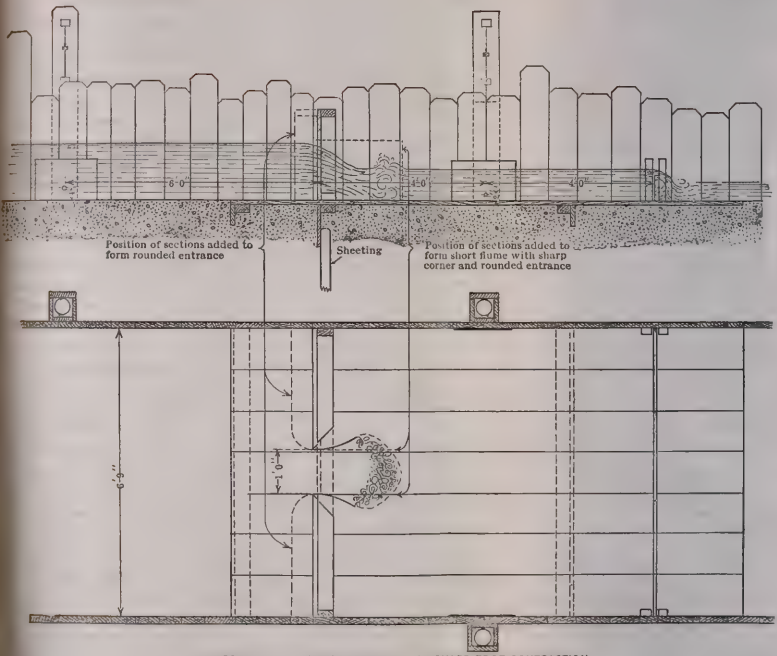
into the river. Of this drop, about 1½ ft. was available for experimental purposes.

The quantity of water used in the experiments was controlled by a by-pass in the side of the channel, about 40 ft. from its upper end. This consisted of a rectangular opening through which the excess flow was discharged back into the river, the quantity being regulated by small stop-logs. Forty feet below the regulator was the weir for measuring the discharge through the experimental contractions, which were built in the channel 86 ft. farther down stream.

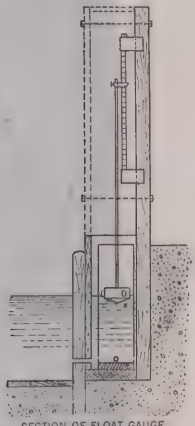
Measuring Weir and Float Gauges.—The measuring weir was of the Cippoletti type, with a crest length of 2 ft. The dimensions of its setting were made as nearly as possible like those of the weir on which experiments* were made by Messrs. W. G. Steward and J. S. Longwell, Jun. Am. Soc. C. E., in order that the coefficients derived by them might be used in computing the discharge. The weir bulkhead was of wood, while the crest was formed of heavy galvanized iron, cut to a straight, smooth edge. The weir was in place several months, and the swelling of the wood and consequent change of crest elevation was complete, before any discharge measurements were made. As in the setting used by Messrs. Steward and Longwell, the crest of the weir was 1 ft. above the bottom of the channel of approach, and the head was measured at a point 10 ft. up stream from the bulkhead.

Float gauges of unusual design were used to measure the head. (Plate XVIII.) They were set in a wooden pit, 6 by 6¾ in., set outside the channel and connected to it through the sheeting by a piece of ⅜-in. pipe, 3 in. long, ending flush with the side of the channel. In order that no cross-currents might be created near the end of the pipe by the irregularities of the sheeting, a piece of galvanized iron was placed over the side of the channel at this point, with a hole drilled through it where it covered the end of the pipe. Inside, and resting on the bottom of the pit, was a cylindrical tube of galvanized iron, 5 in. in diameter, open at the top. In the sides of this cylinder, 1 in. above the bottom, were four holes, ⅜ in. in diameter, spaced equally around the circumference. This tube served as a guide for the float, which was cylindrical in shape, 4 in. in diameter and 2 in. high, with a conical bottom, to prevent air being entrapped beneath it thus alter-

* *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1045.



HORIZONTAL AND VERTICAL SECTIONS OF SHARP EDGE CONTRACTION



SECTION OF FLOAT GAUGE

ing its buoyancy. On the sides of the float were three fins, ½ in. wide, extending out radially to prevent it from rubbing on the side of the tube. Connected to the float by a small brass rod, was a pointer which moved over a steel tape graduated to hundredths of a foot, the tape serving the double purpose of a scale for reading the position of the float and as a guide for the pointer. The gauges were fitted with a wooden box-like cover which was bolted and locked on them when not in use, to prevent their being tampered with. When the experiments were undertaken, it was proposed to place the cylinder and float directly in the channel. This was found to be impracticable, on account of cross-currents, and they were enclosed, therefore, in boxes and placed outside the channel, connected to it with pipes as described.

Float gauges of a somewhat similar pattern, in which the float moves in a cylinder which is connected at the bottom directly to the side of the channel, have been used for several years in the Hydraulic Laboratory of Cornell University, and have been found to be very satisfactory devices for measuring head, particularly if there is considerable wave action. The float gauge has several advantages over the hook gauge for reading water levels. Readings may be taken more rapidly with it; and instantaneous readings, maximum and minimum, and the character of surges can be readily observed. The writer believes that the accuracy obtainable under ordinary conditions is equal to, and, in disturbed water, exceeds that which it is possible to obtain with a hook gauge.

Contractions.—A section of the sharp edge contraction is shown on Plate XVIII, and Fig. 3 shows the flow of water through it. The contraction consisted of two vertical walls, ending with sharp vertical edges formed of galvanized iron, extending out at right angles from the sides of a rectangular channel. The bottom of the channel at this point was formed of planed boards, with cut-off walls below to prevent seepage under the contraction. The height of the water surface above the contraction was measured by one float gauge, 6 ft. above the most contracted section, and the down-stream height by two gauges, one on each side of the flume, 4 ft. below this point. The height of the water surface below the contraction was controlled by a series of small stop-logs, 4 ft. below the down-stream gauges. It was first planned to place this regulator at a considerable distance below the contraction, but when so placed an unstable condition of the jet resulted, which neces-

sitated placing the stop-logs near the contraction. This condition of the jet is described in full in Appendix II. The curved contraction was formed by placing attachments of galvanized iron, curved to a radius of 0.5 ft. above the sharp edge contraction, as shown on Plate XVIII and Fig. 4.

Short Flumes.—The short flume with sharp corner entrance was formed by placing two galvanized iron sheets, 2 ft. long, below the sharp edge contraction, as shown on Plate XVIII. These sheets were fastened securely to the bottom of the channel and braced at the top to hold them in position. The short flume with rounded entrance was formed by placing these pieces below the contraction with rounded entrance.

Expanding Flume.—Plate XXIV shows the cross-section of the expanding or Venturi flume. This flume had sheet-iron sides, tacked to a wooden form which was cut accurately to the desired shape. The entrance was formed by curves of 0.5 ft. radius. Below these, the sides were parallel, 1.0 ft. apart, for 2.0 ft., at which point they began to diverge along the parabolic curve, $y = 0.0178\ x^2$, reaching a distance of 3.0 ft. apart at the lower end of the flume, the total length being 10.0 ft. The flume was set in the channel and connected at the upper end with the same bulkhead that was used for the sharp edge contraction. The bottom of the flume was not exactly level, the down-stream end being slightly higher than the up-stream end. Practically all the slope was in the expanding section of the flume and probably had little, if any, effect on the results.

Point Gauge.—For the determination of the elevation of the water surface in the short and expanding flumes, a point gauge was used. The rod was graduated to hundredths of a foot and moved vertically past a vernier, from which the thousandths of a foot could be read. The rod and vernier were conveyed on a carriage which moved across the channel on a beam, at the ends of which were bearing plates clamped to the framing of the sides of the flume. The position occupied by the bearing plates at each measuring section was marked on the framing of the flume, in order that the beam could be placed in exactly the same position for each reading at that section.

Experimental Methods.

Weir and Float Gauges.—Two methods of determining the gauge zero of the weir were used. In the first of these, readings were taken

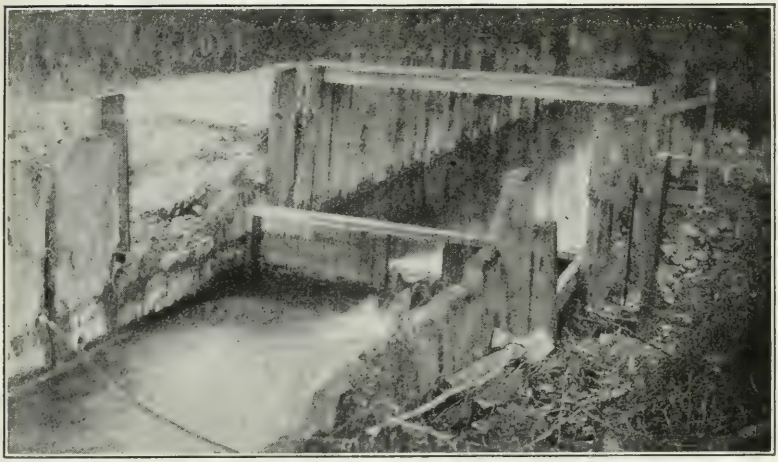


FIG. 3.—ONE-FOOT SHARP EDGE CONTRACTION, SHOWING FLOAT GAUGES.

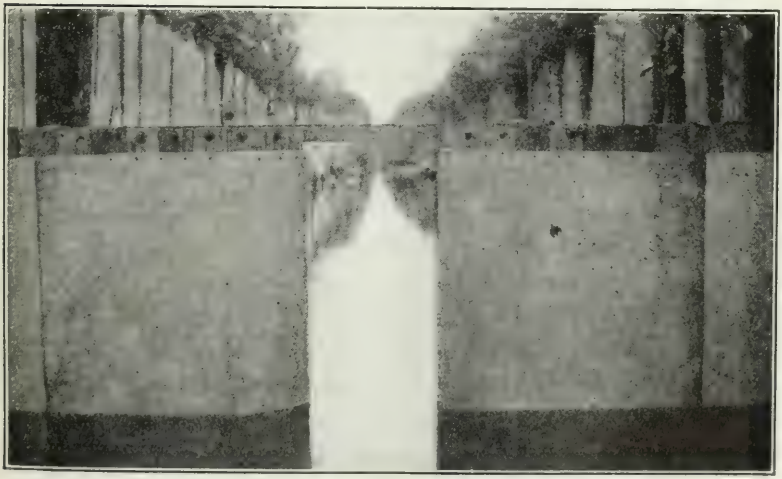


FIG. 4.—CONTRACTION WITH ROUNDED EDGES, UP-STREAM SIDE.

with an engineer's level and rod at several points on the crest of the weir, and on the point of a stationary hook which was fastened to the side of the channel near the float gauge. This hook had a sharp point like an ordinary hook gauge, but was constructed of much heavier material, in order to carry the weight of the level rod without deflection. The lower end of the channel was closed, and water was admitted very slowly until the surface rose slightly above the point of the hook. At the instant the surface reached the point of the hook, a reading of the float gauge was taken. Water was then discharged slowly until the surface again fell below the point, another reading of the float gauge being taken at the instant the water surface reached the point of the stationary hook. The mean of several such observations, which never varied more than a few thousandths of a foot, was taken as the reading of the float gauge corresponding to the elevation of the point of the hook. Knowing the difference in elevation between the crest of the weir and the point of the hook, as determined by level-rod readings, the reading of the float gauge corresponding to the elevation of the weir crest could be readily determined.

The second method of determining the weir gauge zero was by means of the point gauge. Water was ponded in the experimental channel to a sufficient depth to cover the weir crest. The difference in elevation between the crest of the weir and the water surface in the channel was measured with the point gauge at several points along the weir crest, readings of the float gauge being noted at the same time. From these, the float-gauge reading corresponding to the elevation of the weir crest was determined. Both these methods were also used in determining the gauge reading equivalent to the bottom of the contractions.

Contractions.—For the experiments on the contractions with sharp edges, and on the contraction with the rounded entrance, the procedure was the same. Each run consisted of five observations of head on the measuring weir, five on the gauge above the contraction, and ten on each of the gauges below the contraction. The readings on the weir and up-stream gauge varied little during a run, but as a result of the turbulent water below the contraction, considerable fluctuation occurred in the down-stream gauge readings, and, therefore, more observations were required to arrive at an accurate mean. The time required for a run was about 30 min.

Short and Expanding Flumes.—The width of the short and expanding flumes was measured at several elevations at each observation section, by large inside calipers. From these measurements a table was prepared, showing the area of the cross-section of the flume at each observation section for the various depths. To refer the observations on the water surface with the point gauge, the readings of the float gauges, and the elevation of the bottom of the flume at each station, to a common datum, the procedure was as follows: Water was ponded in the flume, the beam of the point gauge was clamped in the proper position, and readings were taken on the water surface and bottom of the flume, at each measuring station, at the same points in the cross-section, which were used later in the flow observations. Simultaneous observations of the water surface elevation, as recorded by the float gauges, were also made. The difference, at any measuring station, between the mean of the readings on the water surface and the mean of those on the bottom, gave the mean depth of water at that section. Knowing the float-gauge readings, as observed for this water surface elevation, the gauge reading corresponding to the bottom of the flume was readily computed. In like manner, the reading of the float gauge equivalent to the mean height of the point-gauge beam was determined for each measuring station.

A run consisted of observations of discharge, depth above and below the contraction, and water surface elevation at the various observing stations. Readings of the weir gauge and the gauges above and below the flumes, for determining the values of the discharge, Q, and of the depths, D_1 and D_3, were made after the measurement of the elevation of the water surface at each station. These stations were spaced along the flume at such distances apart that the profile of the mean water surface could be determined accurately. This was approximately 0.25 ft. in the narrow section of the flumes, but farther apart in the wider portions. At each measuring section in the narrower portions, readings of the water surface elevation were taken at each side and at every 0.2-ft. point between, although, in some cases, they were taken every 0.1 ft. In the lower 2½ ft. of the expanding flume, readings were usually taken 0.3 ft. apart.

Computations.

In the computations for these experiments, all distances were taken to the nearest thousandth of a foot, discharge to the nearest thousandth

of a cubic foot per second, and the coefficients were computed to three decimal places. The multiplications were performed with a Thacher slide-rule or a five-place logarithmic table.

Weir.—The discharge over the weir was computed by the formula:

$$Q = C\,L\left(H + \frac{V^2}{2\,g}\right)^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The values of C were taken from a curve showing the relation of the head to the value of the coefficient, as obtained from the experiments of Messrs. Steward and Longwell previously referred to, on a 2-ft. Cippoletti weir. The length of the crest, L, was obtained by accurate measurements, and H was determined by subtracting the gauge reading corresponding to the weir crest elevation from the reading of the water surface above the weir, corrected for velocity of approach. On all experiments except those on the expanding flume, the mean of the weir gauge readings was used in computing the discharge. For the expanding flume experiments, the discharge corresponding to each gauge reading was computed. The values of the velocity of the approach head, $\frac{V^2}{2\,g}$, were read from a curve computed to show its magnitude for the various values of H.

Contractions.—For the contractions, the equation for the computation of the coefficients in the d'Aubuisson formula was:

$$C_{dA} = \frac{Q}{W_2\,D_2\sqrt{2\,g\;(D_1 - D_2)}} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The quantity, D_1, the total depth above the contraction, was determined from the mean of the up-stream gauge readings, by subtracting from this mean the reading of the gauge corresponding to the elevation of the bottom of the contraction. To this was added the velocity of approach, $\frac{V_1^2}{2\,g}$, V_1 being computed by dividing the measured discharge by the area of the approach channel at the depth, D_1. The value of the width of the contraction, W_2, was determined by measurement with a scale or caliper. The depth at the most contracted section, D_2, was not measured, but, as has been previously noted, the values of D_3, the depth below the contraction, which were determined from the readings of the down-stream gauges, were used in place of D_2, in computing the coefficients by the various formulas.

The computation of the coefficient by the Weisbach formula:

$$C_W = \frac{Q}{W_2 \sqrt{2g} \left(\frac{2}{3} H_2^{\frac{3}{2}} + D_2 H_2^{\frac{1}{2}} \right)} \quad \ldots\ldots\ldots\ldots (7)$$

was made in a manner similar to that for the d'Aubuisson formula. As the bottom was level, $H_3 = D_1 - D_3$, the measured values of H_3 and D_3 being used in the computations, in place of H_2 and D_2, respectively.

Short Flumes.—The computations of the results of the experiments on the short flumes were made in a manner similar to those for the contractions. The same formulas were used, but the coefficients were worked out for each measuring station along the length of the flume, the value of D_2 being the mean depth, and that of W_2 the mean width, as measured for that station. The discharge throughout the run was practically constant, and the mean discharge, therefore, was used for Q. As the bottom was level, $H_2 = D_1 - D_2$. In computing the coefficients for the points below the end of the short flume, the width, W_2, was assumed to be the same as that of the flume at its lower end. It will be noticed that this is the same, in effect, as the assumption usually made in computing the discharge at bridge openings, and, under certain conditions, may lead to erroneous results, as shown on page 1186.

Expanding Flume.—In the experiments on the expanding flume, the coefficients were worked out for each observation section. During a run, a slight change of discharge took place and, therefore, the discharge at the time the measurement was made at any station was used in the computations for that station. The formulas used for the contractions had to be modified somewhat for the expanding flume, as the bottom was not exactly level and $D_1 - D_2$ was not equal to H_2. The true drop was determined by subtracting the elevation of the water surface in the flume, as determined by the point gauge, from the elevation of the water surface above the contraction, as determined by the float gauge, the latter elevation being corrected for the velocity of approach. The measured depth and width at each station were used for D_2 and W_2, respectively.

Results of Experiments on Contractions.

The results of these experiments are given in numerical form in Tables 1 to 10, inclusive. They are also shown graphically on many

TABLE 1.—Results of Experiments on Contraction with Rounded Edges, 1 Ft. Wide.

Run No.	Depth below.	Drop.	Width.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
1	1.094	0.115	1.002	2.525	0.846	0.791
2	1.042	0.130	1.002	2.659	0.880	0.813
3	0.953	0.172	1.002	2.780	0.875	0.781
4	0.889	0.199	1.002	2.796	0.877	0.763
5	0.838	0.205	1.002	2.682	0.879	0.756
6	1.010	0.127	1.002	2.556	0.883	0.815
7	0.998	0.255	1.002	3.661	0.903	0.772
8	1.132	0.212	1.002	3.704	0.884	0.786
9	1.094	0.220	1.002	3.695	0.895	0.789
10	0.940	0.247	1.002	3.687	0.910	0.757
11	0.876	0.318	1.002	3.662	0.922	0.742
12	0.799	0.453	1.002	4.238	0.980	0.711
13	0.668	0.538	1.001	4.193	1.065	0.693
14	0.733	0.484	1.002	4.148	1.012	0.702
15	0.758	0.570	1.002	4.941	1.074	0.715
16	0.811	0.530	1.002	4.894	1.031	0.718
17	0.889	0.458	1.002	4.752	0.982	0.731
18	0.952	0.393	1.002	4.547	0.948	0.743
19	1.050	0.331	1.002	4.464	0.919	0.759
20	1.118	0.300	1.002	4.464	0 907	0.769
21	0.666	0.496	1.001	3.911	1.038	0.694
22	0.713	0.460	1.002	3.858	0.992	0.694
23	0.786	0.397	1.002	3.773	0.948	0.709
24	0.581	0.188	1.001	1.855	0.917	0.754
25	0.739	0.120	1.002	1.842	0.895	0.808
26	0.636	0.160	1.002	1.822	0.891	0.763
27	0.618	0.319	1.001	2.682	0.957	0.712
28	0.706	0.258	1.002	2.637	0.915	0.735
29	0.656	0.283	1.001	2.606	0.930	0.722
30	0.764	0.217	1.002	2.592	0.906	0.762
31	0.817	0.191	1.002	2.585	0.900	0.779
32	0.574	0.186	1.001	1.802	0.906	0.745

of the diagrams. The consistency of the results is highly satisfactory, considering the conditions. In these experiments, it was necessary to measure the difference in elevation between two water surfaces, one of which was subject to considerable fluctuation. In addition, this difference of water level was small, averaging not more than 0.4 ft. A slight error in the observation of either of the water levels, therefore, would result in a considerable error in the coefficient.

Contraction with Rounded Edges.—The results of the experiments on the contraction with rounded edges, on the basis of the d'Aubuisson and the Weisbach formulas, are given in Table 1, and are shown graphically on Figs. 5 and 6. Each run is plotted, its position showing the relation of D_1, the depth above the contraction, to D_3, the depth below. At each of these points is recorded the values of the coefficient obtained for that run. Lines of equal drop (H_3), are drawn for each 0.1 ft., as well as lines of equal values of the ratio, $\frac{H_3}{D_3}$, for each 0.1.

TABLE 2.—Results of Experiments on Sharp Edge Contraction, 1 Ft. Wide.

Run No.	Depth below.	Drop.	Width.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
1	0.675	0.354	1.000	2.456	0.762	0.565
2	0.598	0.250	1.000	1.747	0.728	0.570
3	0.634	0.339	1.000	2.274	0.768	0.566
4	0.664	0.456	1.000	2.962	0.824	0.564
5	0.655	0.437	1.000	2.815	0.811	0.561
6	0.715	0.590	1.000	3.870	0.878	0.567
7	0.678	0.479	1.000	3.122	0.829	0.563
8	0.742	0.616	1.000	4.080	0.874	0.562
9	0.720	0.573	1.000	3.768	0.861	0.563
10	0.686	0.507	1.000	3.290	0.840	0.562
11	0.699	0.542	1.000	3.499	0.848	0.559
12	0.716	0.579	1.000	3.765	0.862	0.560
13	0.650	0.311	1.000	2.137	0.734	0.557
14	0.638	0.281	1.000	1.973	0.725	0.562
15	0.668	0.392	1.000	2.610	0.778	0.559
16	0.780	0.320	1.000	2.602	0.724	0.565
17	0.814	0.418	1.000	3.218	0.762	0.568
18	0.820	0.436	1.000	3.324	0.765	0.565
19	0.825	0.449	1.000	3.431	0.774	0.568
20	0.850	0.524	1.000	3.940	0.798	0.566
21	0.746	0.185	1.000	1.716	0.667	0.572
22	0.770	0.303	1.000	2.433	0.716	0.567
23	0.789	0.355	1.000	2.778	0.736	0.567
24	0.738	0.159	1.000	1.558	0.660	0.577
25	0.760	0.251	1.000	2.122	0.695	0.569
26	0.752	0.210	1.000	1.886	0.682	0.575
27	0.851	0.501	0.996	3.809	0.791	0.568
28	0.958	0.442	0.996	3.818	0.750	0.574
29	0.952	0.417	0.996	3.640	0.742	0.574
30	0.842	0.476	0.996	3.649	0.787	0.571
31	0.932	0.382	0.996	3.380	0.735	0.577
32	0.923	0.362	0.996	3.218	0.725	0.575
33	0.970	0.444	0.996	3.880	0.752	0.576
34	0.928	0.323	0.996	2.977	0.706	0.574
35	0.920	0.296	0.996	2.792	0.698	0.574
36	0.917	0.269	0.995	2.618	0.690	0.577
37	0.940	0.336	0.995	3.114	0.716	0.578
38	0.856	0.185	0.994	1.906	0.649	0.567
39	0.906	0.248	0.994	2.449	0.680	0.575
40	0.885	0.217	0.994	2.219	0.675	0.580
41	0.899	0.184	0.993	2.020	0.657	0.579
42	0.776	0.388	0.993	2.924	0.760	0.570
43	0.592	0.215	0.993	1.558	0.712	0.574
44	0.900	0.152	0.993	1.821	0.652	0.585

It will be noticed that the coefficients for this contraction, according to the Weisbach formula, vary from 0.693 to 0.815, while those of the d'Aubuisson formula vary from 0.846 to 1.074. It would seem that neither formula is a rational expression for the true condition of flow, since with the rounded edges there would be no contraction of the jet, and from the coefficient for orifices with rounded edges, one would expect that the coefficient in the true formula for this contraction would be nearly unity. The d'Aubuisson coefficients fit this condition more closely than those of the Weisbach formula. Some of the d'Aubuisson coefficients, however, have values greater than unity,

TABLE 3.—Results of Experiments on Sharp Edge Contraction, 2 Ft. Wide.

Run No	Depth below.	Drop.	Width.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
1	1.018	0.248	2.000	5.780	0.710	0.611
2	0.816	0.341	2.000	5.830	0.762	0.596
3	0.912	0.295	2.000	5.820	0.732	0.602
4	0.959	0.274	2.000	5.771	0.716	0.602
5	0.862	0.314	2.000	5.729	0.739	0.595
6	0.784	0.306	2.000	5.231	0.752	0.596
7	1.084	0.177	2.000	4.981	0.680	0.614
8	0.718	0.321	2.000	4.990	0.764	0.589
9	1.026	0.145	2.000	4.130	0.659	0.602
10	0.700	0.256	2.000	4.130	0.726	0.584
11	0.787	0.220	2.000	4.120	0.695	0.586
12	0.881	0.184	2.000	4.130	0.681	0.598
13	0.977	0.153	2.000	4.120	0.672	0.608
14	0.908	0.120	2.000	3.303	0.654	0.601
15	0.672	0.190	2.000	3.336	0.710	0.597
16	1.026	0.184	2.000	4.752	0.673	0.600
17	0.968	0.201	2.000	4.752	0.682	0.599
18	0.894	0.225	2.000	4.743	0.697	0.597
19	0.832	0.252	2.000	4.733	0.706	0.588
20	0.786	0.273	2.000	4.743	0.720	0.584
21	0.853	0.115	1.997	3.030	0.654	0.600

TABLE 4.—Results of Experiments on Short Flume with Rounded Entrance.

Position.	Width.	Area.	Drop.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
Run No. 1:						
0.50	0.975	1.106	0.182	4.267	1.126	1.020
1.00	1.000	0.926	0.390	4.267	0.919	0.718
1.50	1.000	0.843	0.469	4.267	0.921	0.672
1.75	1.000	0.818	0.497	4.267	0.922	0.656
2.00	0.995	0.814	0.501	4.267	0.922	0.653
2.17	0.990	0.826	0.486	4.267	0.924	0.662
2.67	*0.990	0.862	0.441	4.267	0.929	0.691
Pool	*0.990	0.849	0.453	4.267	0.930	0.684
Run No. 2:						
0.50	0.975	1.167	0.212	4.735	1.098	0.982
1.00	1.000	0.997	0.405	4.735	0.930	0.696
1.50	1.000	0.913	0.493	4.735	0.921	0.677
1.75	1.000	0.876	0.532	4.735	0.923	0.657
2.00	0.995	0.868	0.540	4.735	0.924	0.653
2.17	0.990	0.866	0.538	4.735	0.928	0.655
2.67	*0.990	0.883	0.512	4.735	0.934	0.672
Pool	*0.990	0.787	0.508	4.735	1.052	0.733
Run No. 3:						
0.50	0.975	0.954	0.187	3.537	1.068	0.947
1.00	1.000	0.812	0.356	3.537	0.910	0.704
1.50	1.000	0.734	0.427	3.537	0.918	0.662
1.75	1.000	0.724	0.440	3.537	0.918	0.653
2.17	0.990	0.740	0.420	3.537	0.919	0.665
2.67	*0.990	0.806	0.345	3.537	0.931	0.722
Pool	*0.990	0.790	0.362	3.537	0.967	0.709

* Assumed to be the same as the lower end of the flume.

TABLE 5.—Results of Experiments on Short Flume, with Sharp Corner Entrance.

Station.	Width.	Area.	Drop.	Discharge.	Coefficients: d'Aubuisson.	Coefficients: Weisbach.
Run No. 1:						
0.50	1.000	0.787	0.378	3.185	0.823	0.621
1.00	1.000	0.626	0.537	3.185	0.865	0.550
1.25	1.000	0.588	0.578	3.185	0.888	0.536
1.50	0.995	0.590	0.574	3.185	0.888	0.538
1.67	0.990	0.598	0.559	3.185	0.888	0.545
2.17	*0.990	0.733	0.421	3.185	0.815	0.602
Pool	*0.990	0.731	0.424	3.185	0.834	0.600
Run No. 2:						
0.50	1.000	0.587	0.333	2.212	0.816	0.590
1.00	1.000	0.475	0.444	2.212	0.871	0.536
1.25	1.000	0.476	0.446	2.212	0.868	0.534
1.67	0.990	0.586	0.326	2.212	0.824	0.599
2.17	*0.990	0.705	0.206	2.212	0.862	0.720
Pool	*0.990	0.655	0.256	2.212	0.832	0.658

* Assumed to be the same as the lower end of the flume.

TABLE 6.—Results of Experiments on Expanding Flume, Run No. 1.

Station.	Bottom Elevation.*	Width.	Area.	Drop.	Discharge.	Coefficients: d'Aubuisson.	Coefficients: Weisbach.
0.50	0.007	0.996	0.882	0.163	2.967	1.038	0.926
0.75	0.000	0.987	0.792	0.255	2.974	0.927	0.765
1.00	0.001	0.984	0.720	0.323	2.967	0.904	0.698
1.25	0.001	0.988	0.684	0.362	2.959	0.896	0.664
1.50	0.008	0.987	0.639	0.400	2.959	0.912	0.646
1.75	0.008	0.992	0.626	0.416	2.959	0.913	0.635
2.00	0.006	0.988	0.616	0.426	2.974	0.922	0.634
2.25	0.010	0.985	0.603	0.436	2.990	0.936	0.634
2.50	0.009	0.979	0.596	0.442	2.995	0.942	0.633
3.00	0.015	0.991	0.590	0.452	3.000	0.942	.....
3.50	0.008	.....	.....	.....	.....	.....	.....
4.00	0.015	1.055	0.569	0.509	3.006	0.923	.....
4.50	0.021	.....	.....	.....	.....	.....	.....
5.00	0.022	.....	.....	.....	.....	.....	.....
5.50	0.024	1.306	0.532	0.633	3.010	0.886	.....
6.00	0.027						
6.50	0.023			6.00 to 7.00 Jump.			
7.00	0.048						
7.50	0.045						
8.00	0.057	2.095	1.320	0.378	3.014	0.463	.....
8.50	0.044	.....	.....	.....	.....	.....	.....
9.00	0.061	2.430	1.567	0.359	3.018	0.401	.....
10.00	0.085	.....	.....	.....	.....	.....	.....

* Above the lowest point of the bottom.

TABLE 7.—Results of Experiments on Expanding Flume, Run No. 2.

Station.	Width.	Area.	Drop.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
0.50	0.996	0.898	0.169	3.077	1.039	0.924
0.75	0.986	0.814	0.253	3.093	0.941	0.782
1.00	0.984	0.750	0.316	3.101	0.916	0.718
1.25	0.988	0.705	0.365	3.109	0.910	0.678
1.50	0.987	0.670	0.396	3.115	0.921	0.663
1.75	0.992	0.651	0.422	3.121	0.920	0.645
2.00	0.988	0.632	0.442	3.126	0.927	0.635
2.25	0.985	0.621	0.448	3.126	0.937	0.635
2.50	0.980	0.624	0.442	3.126	0.939	0.640
3.00	0.991	0.617	0.450	3.126	0.941	.....
3.50	1.017	0.630	0.462	3.126	0.910	.....
4.00	1.055	0.665	0.443	3.123	0.879	.....
4.50 to 5.50 Jump						
6.00	1.419	1.175	0.232	3.120	0.687	.....
7.00	1.702	1.477	0.169	3.117	0.640	.....
8.50	2.300	2 001	0.172	3.117	0.468	.....
10.00	2.990	2.521	0.159	3.117	0.386	.....

TABLE 8.—Results of Experiments on Expanding Flume, Run No. 3.

Station.	Width.	Area.	Drop.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
0.50	0.996	0.932	0.166	3.166	1.040	0.930
0.75	0.987	0.855	0.246	3.178	0.934	0.785
1.00	0.984	0.793	0.308	3.190	0.903	0.719
1.25	0.988	0.759	0.348	3.194	0.889	0.683
1.50	0.987	0.728	0.374	3.198	0.895	0.669
1.75	0.992	0.718	0.389	3.202	0.891	0.657
2.00	0.988	0.723	0.386	3.206	0.889	0.658
2.25	0.986	0.741	0.362	3.210	0.897	0.679
2.50	0.982	0.806	0.295	3.214	0.915	0.737
3.00	0.995	0.873	0.231	3.206	0.952	.....
3.50	1.023	0.917	0.216	3.198	0.935	.....
4.00	1.055	0.872	0.279	3.190	0.863	.....
4.50	1.121	0.907	0.288	3.179	0 814	.....
5.00	1.210	1.140	0.152	3.168	0.888	.....
5 50	1.309	1.219	0.160	3.158	0.807	.....
6.00	1.420	1.277	0.188	3.158	0.711	.....
7.50	1.894	1.703	0.168	3.158	0.564	.....
10.00	2.990	2.709	0.119	3.158	0.421	.....

which, as there must be some loss due to friction, would seem to prove that this formula is irrational. A closer study, however, in connection with the conditions of flow existing in those runs which gave coefficients greater than unity, shows that because of the assumptions made in computing the coefficients, these results would be expected.

TABLE 9.—Results of Experiments on Expanding Flume, Run No. 4.

Station.	Width.	Area.	Drop.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
0.50	0.996	1.026	0.144	3.150	1.008	0.923
0.75	0.986	0.958	0.211	3.154	0.893	0.780
1.00	0.985	0.924	0.247	3.158	0.857	0.729
1.25	0.988	0.917	0.258	3.174	0.849	0.716
1.50	0.988	0.932	0.238	3.190	0.874	0.748
1.75	0.991	0.969	0.202	3.198	0.915	0.805
2.00	0.989	1.060	0.109	3.206	1.141	1.069
2.25	0.986	1.069	0.102	3.214	1.173	1.104
2.50	0.984	1.009	0.170	3.222	0.965	0.869
3.00	0.996	0.945	0.241	3.218	0.864	.....
3.50	1.026	0.972	0.251	3.214	0.822	.....
4.00	1.055	1.146	0.105	3.214	1.078	.....
4.50	1.121	1.144	0.163	3.214	0.867	.....
5.00	1.210	1.188	0.198	3.202	0.755	.....
6.00	1.420	1.498	0.117	3.190	0.776	.....
7.50	1.894	1.989	0.103	3.190	0.623	.....
10.00	2.990	3.050	0.093	3.190	0.427	.....

TABLE 10.—Results of Experiments on Expanding Flume, Run No. 5.

Station.	Width.	Area.	Drop.	Discharge.	Coefficients:	
					d'Aubuisson.	Weisbach.
0.50	0.997	1.204	0.118	3.150	0.949	0.892
0.75	0.985	1.156	0.158	3.142	0.852	0.782
1.00	0.986	1.157	0.157	3.134	0.852	0.782
1.50	0.987	1.206	0.094	3.118	1.051	1.000
2.00	0.989	1.159	0.138	3.101	0.898	0.832
2.50	0.985	1.151	0.135	3.083	0.908	0.843
3.50	1.028	1.181	0.146	3.065	0.846	.....
5.00	1.211	1.418	0.100	3.046	0.846	.....
7.00	1.704	1.992	0.069	3.022	0.720	.....
10.00	2.990	3.361	0.067	2.990	0.428	.....

Instead of proving the irrationality of the d'Aubuisson formula, they really tend to substantiate it.

Consider the d'Aubuisson formula:

$$Q = C_{dA}\, W_2\, D_2 \sqrt{2\, g\, H_2}$$

$$= C_{dA}\, W_2\, D_2 \sqrt{2\, g\, (D_1 - D_2)} \qquad (8)$$

Differentiating with respect to D_2 and equating dQ to zero, in order

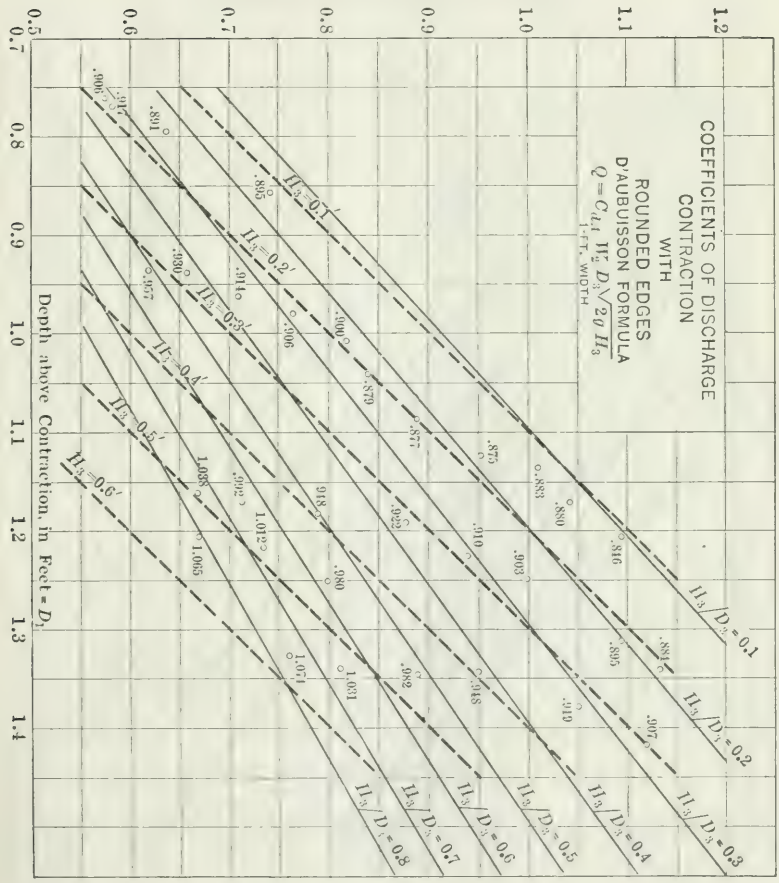

COEFFICIENTS OF DISCHARGE
CONTRACTION
WITH
ROUNDED EDGES
D'AUBUISSON FORMULA
$Q = C_{d,t}\ H_3^{\frac{3}{2}}\ D_3 \sqrt{2g\ H_3}$
1-FT. WIDTH
Depth below Contraction, in Feet = D_3
Depth above Contraction, in Feet = D_3
$H_3/D_3 = 0.1$
$H_3/D_3 = 0.2$
$H_3/D_3 = 0.3$
$H_3/D_3 = 0.4$
$H_3/D_3 = 0.5$
$H_3/D_3 = 0.6$
$H_3/D_3 = 0.7$
$H_3/D_3 = 0.8$
$H_3 = 0.1'$
$H_3 = 0.2'$
$H_3 = 0.3'$
$H_3 = 0.4'$
$H_3 = 0.5'$
$H_3 = 0.6'$

Fig. 5.

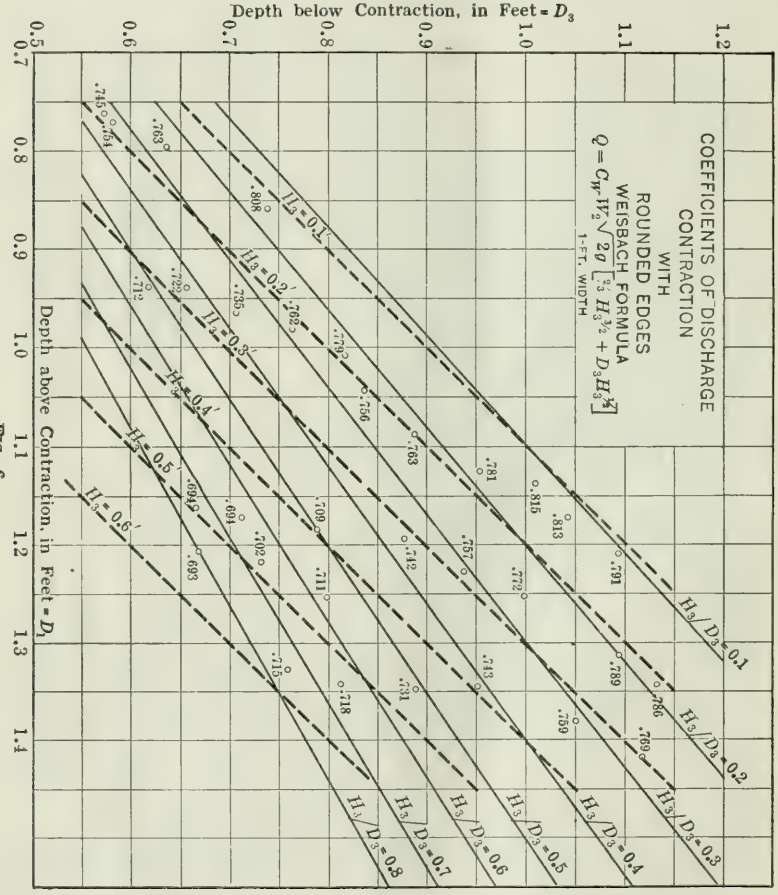

COEFFICIENTS OF DISCHARGE
CONTRACTION
WITH
ROUNDED EDGES
WEISBACH FORMULA
$Q = C_W W_2 \sqrt{2g} \left[\frac{2}{3} H_3^{3/2} + D_3 H_3^{1/2}\right]$
1-FT. WIDTH
Depth below Contraction, in Feet = D_3
Depth above Contraction, in Feet = D_1
$H_3/D_3 = 0.1$
$H_3/D_3 = 0.2$
$H_3/D_3 = 0.3$
$H_3/D_3 = 0.4$
$H_3/D_3 = 0.5$
$H_3/D_3 = 0.6$
$H_3/D_3 = 0.7$
$H_3/D_3 = 0.8$
$H_3 = 0.1'$
$H_3 = 0.2'$
$H_3 = 0.3'$
$H_3 = 0.4'$
$H_3 = 0.5'$
$H_3 = 0.6'$

FIG. 6.

to find the value of D_2 which gives the maximum discharge, it is found that this occurs when D_2 is equal to $\frac{2}{3} D_1$, or when H_2 is equal to $\frac{1}{2} D_2$, or $\frac{1}{3} D_1$. The maximum discharge, therefore, is:

$$Q = C_{dA} W_2 \frac{2}{3} D_1 \sqrt{2 g \frac{D_1}{3}} \quad \ldots\ldots\ldots\ldots\ldots (9)$$

$$= C_{dA} W_2 3.09 D_1^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

In flowing, on a level bottom, from a body of water at one elevation to another at a lower elevation, where D_3 is less than $\frac{2}{3} D_1$, the water tends to assume the condition of maximum discharge, and flow in the same manner as if D_3 were equal to $\frac{2}{3} D_1$. This phenomenon was pointed out by Unwin* and Frizell† and has been described before the Society.‡

From Fig. 5, it will be seen that all those runs which give d'Aubuisson coefficients greater than unity, have $\frac{H_3}{D_3}$ greater than 0.5. This is also shown on Fig. 7, where the d'Aubuisson coefficients are plotted against the ratio, $\frac{H_3}{D_3}$. The flow in these cases, therefore, was similar to the condition of maximum discharge and had a depth in the most contracted section of, approximately, $\frac{2}{3} D_1$. The theoretical discharge according to the d'Aubuisson formula was computed on the assumption that the depth of the stream in the most contracted section was equal to D_3, the depth in the pool below the contraction. For those runs in which D_3 was less than $\frac{2}{3} D_1$, D_3 was less than D_2, and the computed discharge, therefore, was less than the actual discharge, causing coefficients greater than unity. For this reason, the d'Aubuisson formula should not be used where $D_3 < \frac{2}{3} D_1$, unless the value of D_2 is actually measured and used in the computations.

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* *Encyclopedia Britannica*, under "Hydrodynamics".
† "Water Power", by Frizell, pp. 198-200.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 338.

If one computes a coefficient, C_M, showing the ratio of the actual discharge to the "maximum" discharge, $Q = 3.09\ W_2\ D_1^{\frac{3}{2}}$, for those runs in which H_3 is greater than $\frac{1}{2}\ D_3$, it will be found that, in some cases, C_M is greater than unity, or that the actual discharge was greater

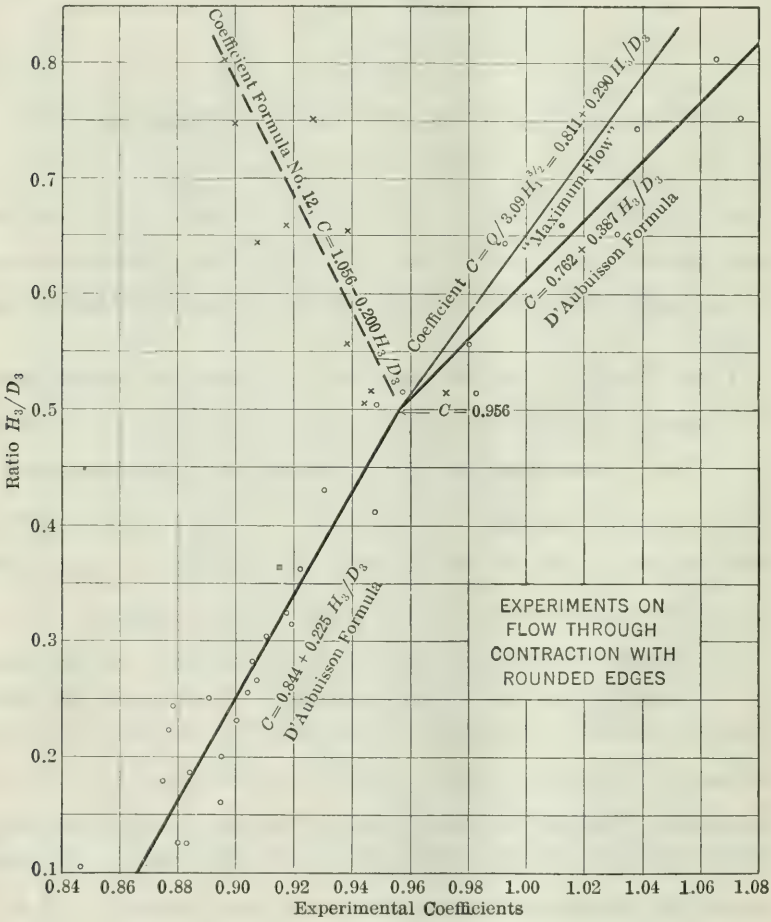


FIG. 7.

than the maximum possible, according to this theory. Here, again, an examination of the condition of flow shows the reason for this discrepancy. The conditions which give rise to the "maximum flow" are shown on Fig. 8, the cross-hatched area representing graphically

the discharge per unit width. In this case, $D_2 = D_3 = \frac{2}{3} D_1$. In those cases where the coefficient, C_M, is greater than unity, D_3 is less than $\frac{2}{3} D_1$, and the conditions of flow are as shown on Fig. 9.

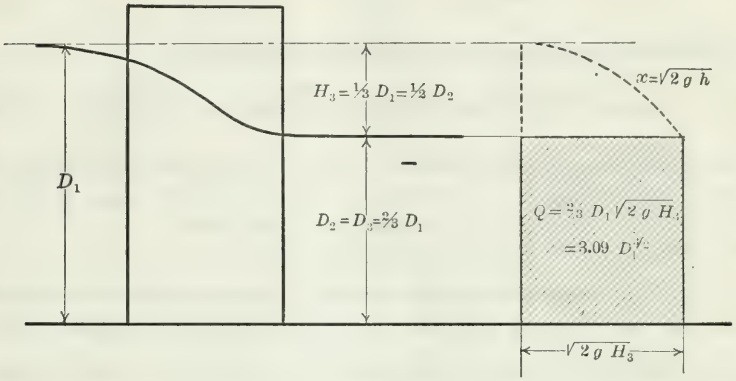


FIG. 8.

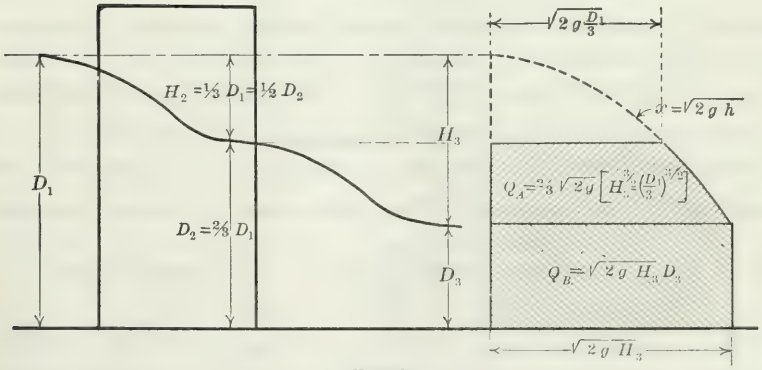

$\sqrt{2 g \frac{D_1}{3}}$
$H_2 = \frac{1}{3} D_1 = \frac{1}{2} D_2$
H_3
$x = \sqrt{2 g h}$
D_1
$Q_A = \frac{2}{3} \sqrt{2 g} \left[H_3^{3/2} - \left(\frac{D_1}{3}\right)^{3/2} \right]$
$D_2 = \frac{2}{3} D_1$
D_3
$Q_B = \sqrt{2 g H_3} D_3$
$\sqrt{2 g H_3}$

FIG. 9.

Under these conditions, all those particles of water which flow through the contraction below the elevation of the down-stream water surface, have a velocity due to the difference of elevation of the water surface above and below the contraction, or $\sqrt{2 g H_3}$. They make up the lower cross-hatched rectangle of Fig. 9, with an area of $D_3 \sqrt{2 g H_3}$. Those particles which pass out of the contraction at an elevation above

the lower pool, have a velocity, depending on their elevation, varying from $\sqrt{2\,g\,\frac{D_1}{3}}$ to $\sqrt{2\,g\,H_3}$, the total discharge through this section being

$$W_2\,\frac{2}{3}\,\sqrt{2\,g}\,\left[H_3^{\frac{3}{2}} - \left(\frac{D_1}{3}\right)^{\frac{3}{2}}\right] \ldots\ldots\ldots\ldots (11)$$

and the total discharge through the contraction being

$$Q = C\,W_2\,\sqrt{2\,g}\,\left[D_3\,H_3^{\frac{1}{2}} + \frac{2}{3}\left(H_3^{\frac{3}{2}} - \left\{\frac{D_1}{3}\right\}^{\frac{3}{2}}\right)\right] \ldots . (12)$$

This formula, as will be shown later, is only applicable where the flowing stream has an opportunity to expand laterally when, or shortly after, it reaches the point where its depth is $\frac{2}{3}\,D_1$.

Fig. 7 shows graphically the results of the experiments on the contraction with curved entrance, the values of the coefficients being plotted against the values of the ratio, $\frac{H_3}{D_3}$. It will be seen that the values of the coefficient seem to be dependent on this ratio, that is, all cases having the same value of $\frac{H_3}{D_3}$ have the same coefficient. This fact makes it possible to obtain a simple formula for the coefficient of these experiments, as it combines the two variables, D_1, the depth above, through the relation $D_1 = H_3 + D_3$, with D_3, the depth below, into a single variable. The results show that the coefficients in the d'Aubuisson formula increase at a uniform rate, with respect to the values of $\frac{H_3}{D_3}$, until the depth of maximum flow is reached, or where $H_3 = 0.5\,D_3$. For the values of $\frac{H_3}{D_3}$ up to 0.5, the coefficient can be expressed by the formula, $C = 0.844 + 0.225\,\frac{H_3}{D_3}$. Above this critical depth, the coefficient increases at a rapid rate, soon becoming greater than unity, for the reason already explained. This portion also seems to follow a straight line law, with a formula, $C = 0.762 + 0.387\,\frac{H_3}{D_3}$. If the formula, $Q = C_M\ 3.09\ W_2\ D_1^{\frac{3}{2}}$, is used for the discharge, where $\frac{H_3}{D_3} > 0.5$, the coefficients follow a line as shown, which differs little from that of the d'Aubuisson formula. This coefficient also

becomes greater than unity, as previously explained. The experimental points were not put on the diagram, in order to avoid confusion, as many of them fall close to those of the d'Aubuisson formula. This curve has a formula, $C = 0.811 + 0.290 \frac{H_3}{D_3}$. The coefficients determined by Formula (12), for runs where $\frac{H_3}{D_3} > 0.5$, also seem to follow a straight line, decreasing uniformly as the ratio, $\frac{H_3}{D_3}$, increases, according to the equation, $C = 1.056 - 0.200 \frac{H_3}{D_3}$. Algebraically, if the formula for C_{dA} follows a straight line, those for C_M and the coefficient for Formula (12) do not necessarily follow straight lines. Practically, however, the straight line relation seems to fit all three cases.

From the results of the experiments on the short flume with rounded entrance and on the expanding flume, it seems probable that the difference between the theoretical and actual discharge of the contraction with rounded edges is due to a loss of the nature of an impact rather than a friction loss. The increase of the coefficient, C_{dA}, as the value of $\frac{H_3}{D_3}$ increases, shows that this loss becomes a relatively smaller portion of the head, as the value of $\frac{H_3}{D_3}$ increases. The variability of the coefficient may be in part due to a slight recovery of the elevation head of the water after passing through the contraction. Such a recovery occurred in the short flume with rounded entrance, as will be explained later, but the measurements on the contraction with rounded edges were not of such a nature that its presence, if it existed, could be shown. The phenomenon of change from stream line to turbulent flow may also be a contributing factor. It has been suggested that this change of state of motion of the water is responsible for certain phenomena of flow over a sharp crested weir at low heads.*

The decrease in the coefficient for Formula (12), where $\frac{H_3}{D_3} > 0.5$, may be due to an assumption on which the formula is based, which may be only approximately true, namely, that when $\frac{H_3}{D_3} > 0.5$, H_2, the water

* "Control of Water", by Parker, p. 22.

surface drop, in flowing through the contraction, is always equal to $\frac{1}{3} D_1$. This is probably true when $D_3 = \frac{2}{3} D_1$, but a somewhat different set of conditions exist when $D_3 < \frac{2}{3} D_1$, which may cause the drop of the water surface to follow a slightly different law. Just what this law is would probably have to be determined by measurement. H_2 would probably not be a constant proportion of D_1, but would probably start at $\frac{1}{3} D_1$, when $\frac{H_3}{D_3} = 0.5$ and, as the decreasing coefficients seem to indicate, become a larger part of D_1 as the ratio, $\frac{H_3}{D_3}$, increases.

Sharp Edge Contractions.—The results of the experiments on the sharp edge contractions are given in Tables 2 and 3. It will be noticed that for both the 1 and 2-ft. widths, the values of the coefficients are practically constant for the Weisbach formula, while for the d'Aubuisson formula, they vary over quite a range. Moreover, the value of the coefficient in the Weisbach formula is in fair agreement with the values found on the sharp edge weir and orifice. If the conditions of flow at these contractions are examined, it is evident that, for this case, the Weisbach, rather than the d'Aubuisson, formula is the rational one. The reason for this is evident from a study of Fig. 2. In the case of the contraction with rounded edges, where $H_3 < \frac{1}{2} D_3$, all the drop takes place within the contraction. Each particle of water, therefore, acquires a velocity depending on the difference of the water level above and below the contraction, or $\sqrt{2 g H_3}$, and the total discharge is, therefore, $Q = W_2 D_3 \sqrt{2 g H_3}$. In the sharp edge contraction, however, the greater part of the drop takes place after the water has passed through the contraction. (Fig. 3.) Each particle above the tail-water level, as it passes through the contraction, has a velocity in proportion to its drop, or to $\sqrt{2 g h}$, and the discharge per unit width of this portion is:

$$\int_0^{H_3} \sqrt{2 g h} = \frac{2}{3} \sqrt{2 g} H_3^{\frac{3}{2}}.$$

All the particles below the tail-water level have the same velocity, or $\sqrt{2 g H_3}$, and the discharge per unit width of this portion is

$D_3 \sqrt{2\,g\,H_3}$. Combining these two parts, we have the Weisbach formula:

$$Q = C_W\, W_2 \sqrt{2\,g} \left(\frac{2}{3} H_3^{\frac{3}{2}} + D_3\, H_3^{\frac{1}{2}} \right).$$

As H_3 and D_3 were actually measured and used in the computations of the results of the sharp edge contractions, it will be seen that if H_3 and D_3 are used in place of H_2 and D_2, the flow through the sharp edge contraction agrees with the Weisbach theory. The Weisbach formula is essentially the same as the theoretical formula for a submerged weir, and the condition of flow through a sharp edge contraction is similar to that which theoretically takes place over such a weir. In all cases of flow through contractions, the individual water particles follow the Bernouilli theorem, and if the area of the stream from the sharp edge contraction was measured where it reaches the elevation of the down-stream pool, the discharge no doubt would be found to be $Q = A \sqrt{2\,g\,H_3}$, where A represents this area.

The Weisbach coefficients for the sharp edge contractions are not exactly constant, but vary slightly, being larger for the smaller values of $\frac{H_3}{D_3}$. This might indicate that the coefficients for the two portions (Formulas (2) and (3)) have different values, instead of the same value, which is an assumption of the Weisbach formula. The formula on this basis, therefore, would be :

$$Q = W_2 \sqrt{2\,g} \left[C_A \frac{2}{3} H_3^{\frac{3}{2}} + C_B\, D_3\, H_3^{\frac{1}{2}} \right] \ldots\ldots\ldots (14)$$

The coefficients in this formula may be readily found by transferring Formula (14) into

$$Q = \left(\frac{2}{3} C_A \frac{H_3}{D_3} + C_B \right) W_2 \sqrt{2\,g}\, D_3\, H_3^{\frac{1}{2}} \ldots\ldots\ldots (15)$$

It will be recognized that this has the same form as the d'Aubuisson formula, except in this case the coefficient is a variable composed of two parts, one of which is a constant, while the other is a linear function of $\frac{H_3}{D_3}$. If, therefore, the d'Aubuisson coefficients of these experiments are plotted against their respective values of $\frac{H_3}{D_3}$ (Fig. 10), and the straight line most nearly representing the points is drawn, we have directly the

values of C_B and C_A, C_B being the value of the d'Aubuisson coefficient, as shown by this line, when $\frac{H_3}{D_3} = 0$, and C_A being $\frac{3}{2}$ of the slope of the line with respect to the axis along which the values of the ratio, $\frac{H_3}{D_3}$, were plotted.

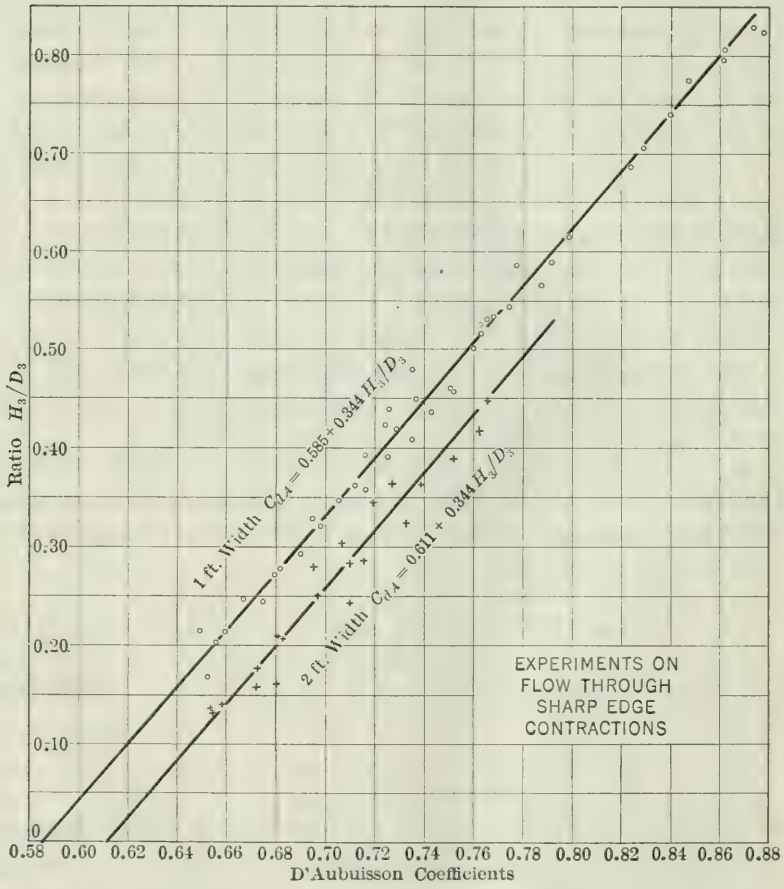


FIG. 10.

Fig. 10 shows the results of this plotting for both the 1-ft. and 2-ft. widths, and the d'Aubuisson forms thus derived for the discharges are:

$$Q = \left[0.585 + 0.344\,\frac{H_3}{D_3}\right] W_2 \sqrt{2g}\, D_3\, H_3^{\frac{1}{2}} \ldots\ldots\ldots (16)$$

for the 1-ft. width, and

$$Q = \left[0.611 + 0.344 \frac{H_3}{D_3}\right] W_2 \sqrt{2g}\, D_3 H_3^{\frac{1}{2}} \ldots\ldots\ldots (17)$$

From these, the coefficients in Formula (14) are, therefore, for the 1-ft. width, $C_B = 0.585$, and $C_A = \frac{3}{2} \times 0.344 = 0.516$, and for the 2-ft. width, $C_B = 0.611$ and C_A also $= 0.516$.

It will be noticed that the coefficient for that part of the flow below the tail-water elevation is larger than that for the part above. The reason for this is probably because the portion below is contracted on two sides only, while that above is contracted on both sides and top, since the upper surface, as it flows through the contraction, has already dropped a certain distance and does not remain at the elevation of the head-water, as assumed in the Weisbach formula. This tends to cause a greater contraction in the upper part of the stream and, therefore, produces a lower coefficient for this part.

The coefficient, C_A, for both widths is found to be the same, but as the range covered by the experiments on the 2-ft. width was small, due to the limitations of the apparatus, the values are not clearly defined, and experiments covering a larger range might show them to differ slightly. The writer believes that the same coefficient in the two cases is the result of a balance of the effects of the contraction on the sides and upper surface of this part of the flow. The effect of the contractions on the sides is less for the larger width, as is shown by the greater coefficient for that width, for the portion of the stream below the tail-water level. On this part only the side contractions are acting. The contraction due to the drop of the water surface before passing between the sharp edges is probably less for the narrow width, since the contraction of the stream at the sides tends to prevent this surface drop from taking place. The effect of these side contractions is greater on the narrower width, since, in this case, they occupy a greater proportion of the width of the opening. The tendency of the contraction on the sides of the 1-ft. width to be larger, and on the top to be smaller, than for the 2-ft. width, probably equalize each other, and give the same coefficients on both widths.

The flow in the section of the contraction above the tail-water level is probably somewhat similar to that through a rectangular weir with

end contractions. The late James B. Francis, Past-President, Am. Soc. C. E., found that the effect of the end contractions was to reduce the effective length of the weir by 0.2 H. His formula, however, if carried to extremes, leads to a negative discharge, and experiments* show that although it is applicable up to a certain point, for widths narrower than this, the end contractions interfere in the center, and the reduction of the effective length becomes less than 0.2 H. The discharge, therefore, tends to become more like that through an orifice. As it seems reasonable that, for a very large width, the discharge of a sharp edge contraction would be practically the same as for one with rounded edges, one would expect that, as the width of the contraction was increased, the coefficient, C_B, would increase, while C_A would decrease.

Different coefficients for the parts of the flow above and below the tail-water level have been found for the flow over a submerged weir. Mr. Francis† found the coefficients for a submerged weir without end contractions to be 0.622 and 0.573 for the part above and below the tail-water, respectively. For the part above, the coefficient for the weir is larger than that of the contraction. This is probably due to the fact that, in the case of the weir, the coefficient expresses the effect of the surface drop only, while, in the case of the contraction, it is due to both the surface drop and the effect of the sides. On the other hand, for the part below the tail-water level, the coefficient for the weir is slightly smaller than for the contractions, indicating that the effect of the sides of the contraction is less than that of the crest of the weir.

General Formula for Contractions.

A general formula for contractions may be developed in a manner similar to Formula (12), by using the water surface drop, H_2, instead of $\frac{1}{3} D_1$. This formula is

$$Q = C_G \; W_2 \sqrt{2g} \left[D_3 \, H_3^{\frac{1}{2}} + \frac{2}{3} \left(H_3^{\frac{3}{2}} - H_2^{\frac{3}{2}} \right) \right] \ldots\ldots (18)$$

When the conditions are such that all the drop takes place after the water passes through the contraction, H_2 is zero, and the formula

* *Cornell Civil Engineer*, April, 1912.

† *Transactions*, Am. Soc. C. E., Vol. XIII (1884), p. 303.

becomes the same as that used in computing the Weisbach coefficients. When the drop takes place within the contraction, H_2 is equal to H_3, and the formula becomes that used in the computation of the d'Aubuisson coefficients. In order for this formula to apply, it is necessary for the flowing stream to be able to expand laterally without hindrance immediately after passing through the narrowest portion of the contraction, which limits the use of the formula to contractions the lengths of which in the direction of flow, are rather short. The reason for this will be shown in the discussion of the results of the experiments on short flumes. It is evident, therefore, that the sharp edge contraction, where very little drop takes place before the water passes between the sharp edges, is the condition most favorable to the application of the Weisbach formula, while the contraction with a gradually decreasing width, where all the water surface drop has taken place before the water passes the narrowest section, is the most favorable for the d'Aubuisson formula. For conditions between, where part of the drop takes place before the water reaches the most contracted section, and part occurs after this point is passed, the general formula applies. The contraction with rounded edges, where $H_3 > \frac{1}{2} D_3$, falls in this latter case, the value of H_2 not being the constant, $\frac{1}{3} D_1$, as in Formula (12), but a variable, as indicated on page 1176.

The coefficient, C_G, of the general formula considers only the contraction at the sides, since the contraction at the upper surface is taken care of by the term, H_2. One would expect, therefore, that this coefficient would have substantially the same value as that for orifices with the same class of edges. Thus, for those cases where the entrance is formed by gradual curves, this coefficient would approach unity, while, for sharp edges, it would approach the coefficient of a sharp edge orifice.

It may be reasonably assumed that, except for the contraction of that part of the stream above the tail-water level, due to the drop of the water surface before passing through the contraction, C_A and C_B in Formula (14) would be equal. If this is true, the term, H_2, in the general formula, accounts for the difference between C_A and C_B, and the coefficient, C_G, of the general formula should be equal to C_B, since

each is the coefficient due to contraction at the sides only. Assuming $C_G = C_B$, by equating Formulas (14) and (18), we have

$$C_G\left[H_3^{\frac{3}{2}} - H_2^{\frac{3}{2}}\right] = C_A H_3^{\frac{3}{2}} \quad \ldots(19)$$

or,

$$\frac{H_2}{H_3} = \left[\frac{C_G - C_A}{C_G}\right]^{\frac{2}{3}} = \left[\frac{C_B - C_A}{C_B}\right]^{\frac{2}{3}} \quad \ldots(20)$$

From these assumptions, it follows, therefore, that the drop in the water surface at the "crest" of the sharp edge contraction is a constant proportion of the total drop. By inserting the values of C_A and C_B in Formula (20), $\frac{H_2}{H_3}$ is found to be 0.240 for the 1-ft. width, and 0.298 for the 2-ft. width. This agrees with the statement on page 1179 that the surface drop is greater for the 2-ft. contraction. In having the surface drop a constant proportion of the total drop, the contraction resembles the weir, the surface drop at the crest of which has been found to be a constant proportion of the head.*

Results on Experimental Flumes.

Short Flume with Rounded Entrance.—The results of the experiments on the short flume with rounded entrance are given in Table 4 and are shown by Figs. 11, 12, and 13.

The author regrets that he was prevented from completing the series of experiments on this flume, which he had planned, but as these few experiments throw considerable light on the theory of flow through a contraction, they will be fully described. The coefficients at the various observing sections for both the d'Aubuisson and Weisbach formulas are plotted at their respective distances from the entrance. A profile is given, showing the mean surface of the water as it flows through the flume. At each point along this profile is also shown the velocity head of the water at that point and the head lost in friction up to that point, which is, of course, the difference between the actual drop of the water and that part of this drop which is transformed into velocity. A line is also drawn showing at all sections the critical depth for a stream flowing with the velocity existing at that section.

An interesting difference between the flow through the short flume with rounded entrance and the contraction with rounded edges will be

* "Expériences Nouvelles sur l'Ecoulement en Deversoir exécutées a Dijon de 1886 a 1895", H. Bazin, 1898. Translated by A. Marichal and J. C. Trautwine, Jr., Assoc. Am. Soc. C. E., in the *Proceedings* of the Engineers' Club of Philadelphia.

noticed. At the end of the entrance section of the short flume, the surface drop is considerably less than the maximum drop in the flume. Although no actual measurements were made of the surface drop in the contraction with rounded edges, except in a few runs, no material drop was observed in the water surface after passing through the contraction. A possible cause of this difference is the constraining effect

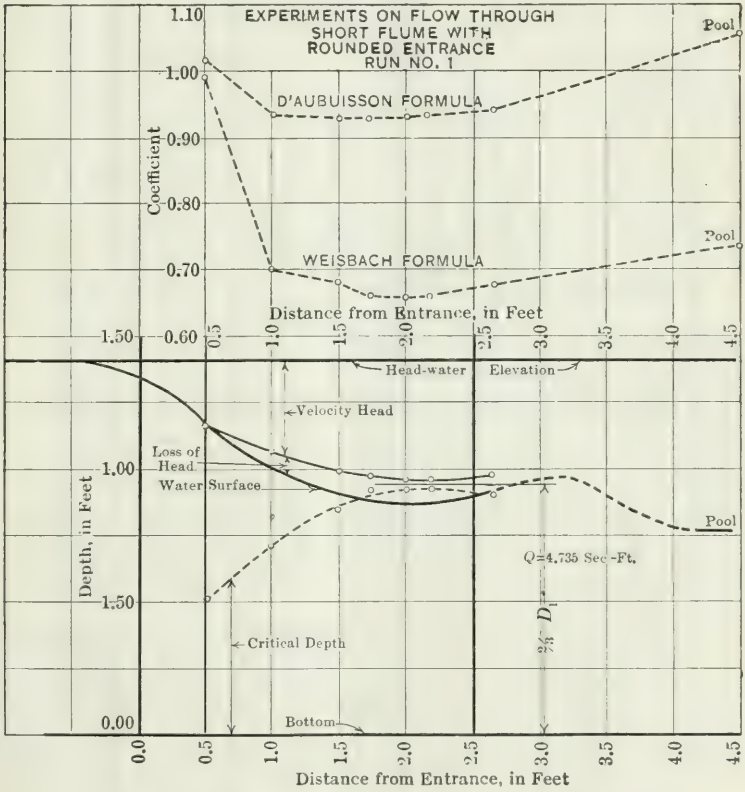


FIG. 11.

of the sides of the flume. In flowing through the contraction with rounded edges, or the entrance section of the flume, the water surface drops, and the water particles near it flow in a downward direction. Those near the bottom, however, flow horizontally. Because of their inertia, the particles tend to maintain their direction after passing through the converging portion. The path of the particles from above

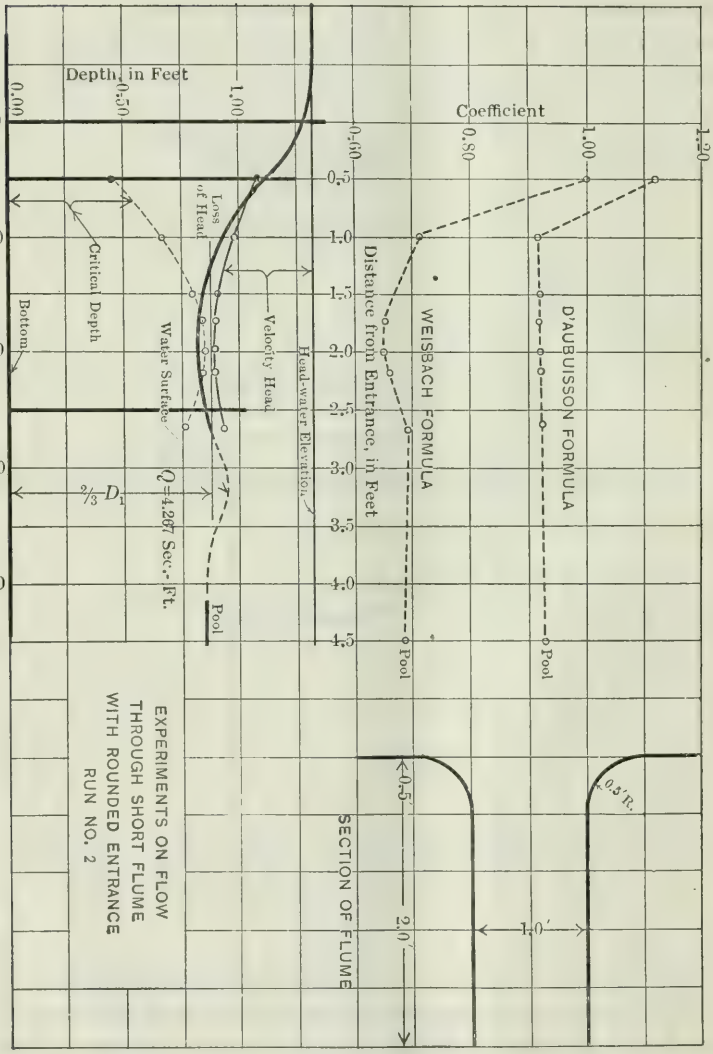

Depth in Feet
Coefficient
Distance from Entrance, in Feet
Distance from Entrance, in Feet
WEISBACH FORMULA
D'AUBUISSON FORMULA
Pool
Pool
Pool
Loss of Head
Velocity Head
Head-water Elevation
Critical Depth
Bottom
Water Surface
$\frac{2}{3} D_1$
$Q = 4.267$ Sec.-Ft.
0.5 R.
0.5'
2.0'
1.0'
SECTION OF FLUME
EXPERIMENTS ON FLOW
THROUGH SHORT FLUME
WITH ROUNDED ENTRANCE
RUN NO. 2

FIG. 12.

intersects that of those from below, and each group tends to be deflected somewhat from the original direction. In the case of the contraction, the particles in the bottom portion are deflected laterally, while those above are given a more nearly horizontal direction. In the flume, however, because of the constraining effect of the sides, the lower

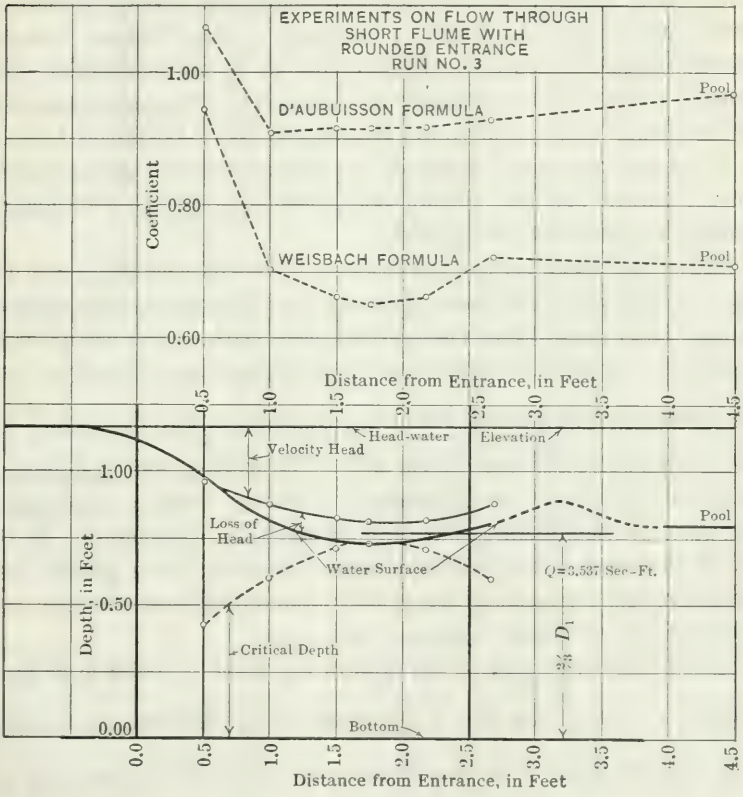


FIG. 13.

particles must continue in a horizontal direction and the particles above, therefore, cannot drop as rapidly as they would if those below could be deflected. The particles above, therefore, gradually acquire a more nearly horizontal flow as they pass down the flume, the lowest point in the water surface being reached some distance down stream from the entrance.

The coefficients of both formulas are found to have high values at the first measuring section of the short flume with rounded entrance. The condition of flow in this case is similar to that for which the general formula for contractions (Formula (18)) was derived, and, therefore, gives a high value of the coefficient when the assumption is made that the total drop is the same as the water surface drop at that point. Neglecting this first measuring point, the d'Aubuisson coefficients average about 0.92, while those of the Weisbach formula average approximately 0.67. Here, again, as in the contraction with rounded edges, one would expect the coefficient of the true formula to have values of nearly unity, which points to the d'Aubuisson formula as being the true one for this case. Experiments on the expanding flume indicate that the d'Aubuisson coefficients for other discharges would not necessarily remain 0.92.

The d'Aubuisson coefficients for the point 2 ft. below the end of the contraction vary, being greater than unity for Run No. 1. As explained on page 1162, the width of the flowing stream at this point was assumed to be the same as that of the lower end of the flume. Since, in Run No. 1, the depth in the pool below the contraction, was less than $\frac{2}{3} D_1$, the theoretical discharge which would result from the assumptions made on the basis of the d'Aubuisson formula, would be considerably less than the maximum possible flow through the contraction. As the actual discharge was very close to this maximum, it was greater than the theoretical discharge as computed by the d'Aubuisson formula, and, therefore, the coefficient is greater than unity.

In all three experiments, the drop in the flume is found to be practically $\frac{1}{3} D_1$. For Run No. 1, the depth of the tail-water is less than $\frac{2}{3} D_1$, and the water in flowing through the flume takes practically the depth which will give maximum discharge, or $\frac{2}{3} D_1$. That it does not take exactly this depth may be accounted for perhaps by the fact that, in spite of the smooth curves of the rounded entrance, there was some disturbance as the water entered the flume, and a considerable entrance loss, as is shown by the loss of head at the 1.0-ft. point. In Runs Nos. 2 and 3, the drop is also practically $\frac{2}{3} D_1$, although D_3 is slightly greater

than D_2. This would indicate a slight recovery of head, or change from velocity to elevation head, of the water after leaving the flume. This seems to have been accomplished in a stationary wave which occurred at the lower end of the flume. Just how much of the velocity head of the water can be recovered in the form of elevation head in this way cannot be determined from these experiments, but, in both cases in which it occurred, the recovery was slight. That the drop of the water surface in the flume should be practically $\frac{1}{3} D_1$ for cases where the depth of the tail-water is much greater than $\frac{2}{3} D_1$, is not to be expected.

In the short flume with rounded entrance, where the water reaches the depth $\frac{2}{3} D_1$ some distance up stream from the lower end, it is not possible for the flow to exceed $3.09\ W_2\ D_1^{\frac{3}{2}}$, as was the case for the contraction with rounded edges, as no expansion of the jet is possible immediately after the stream passes the lowest point within the contraction. The derivation of the theory, according to which the maximum flow is $3.09\ W_2\ D_1^{\frac{3}{2}}$, is based on the Bernouilli theorem, which, for open channels, assumes that each particle of water is subjected to a static pressure equal to the weight of the particles above it, up to the free water surface. When this condition does not exist, the maximum flow does not occur when the drop is $\frac{1}{3} D_1$. When the stream is free to expand laterally, its sides, as well as its upper surface, are at atmospheric pressure, and, therefore, all the particles within it are not subjected to the full static pressure due to the height of the water above them. Just how far up stream the effect of the reduction of pressure on the sides extends, is not easily determined, but it is probably dependent on the magnitude of this reduction. For small models such as those experimented on, it probably causes appreciable effect only a short distance above. From the foregoing, it will be seen that where the stream is free to expand after it passes the narrowest section, as in the contraction with rounded entrance, the discharge can exceed the "maximum", but in the short flume with rounded entrance, where the water surface reaches its lowest point some distance above the end of the flume, the "maximum" cannot be exceeded.

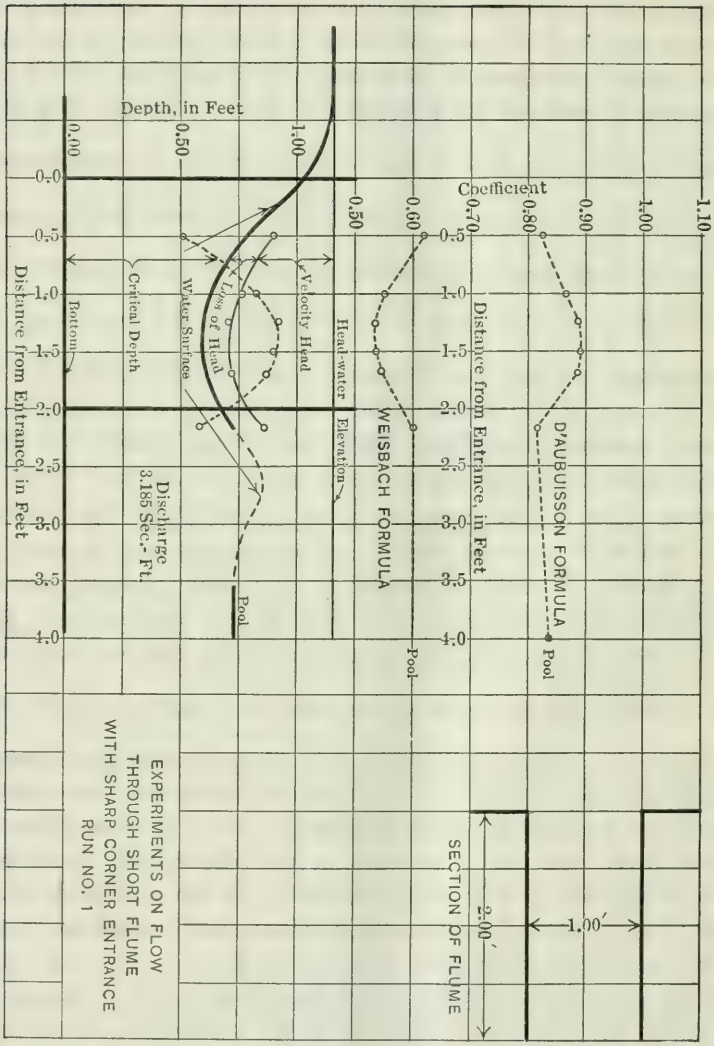

Coefficient
1.10
1.00
0.90
0.80
0.70
0.60
0.50
D'AUBUISSON FORMULA
Pool
WEISBACH FORMULA
Pool
Distance from Entrance, in Feet
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
SECTION OF FLUME
1.00'
2.00'
Depth, in Feet
0.00
0.50
1.00
Velocity Head
Head-water Elevation
Loss of Head
Water Surface
Critical Depth
Bottom
Pool
Discharge 3.185 Sec.-Ft.
EXPERIMENTS ON FLOW THROUGH SHORT FLUME WITH SHARP CORNER ENTRANCE
RUN NO. 1
Distance from Entrance, in Feet
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

FIG. 14.

Figs. 11, 12, and 13 show that the loss of head which occurs in the stream as it flows through the flume increases very little after the drop of the water surface is practically complete. This indicates that the loss took place in the curved entrance, and was, therefore, more of the nature of an entrance or impact loss, than a friction

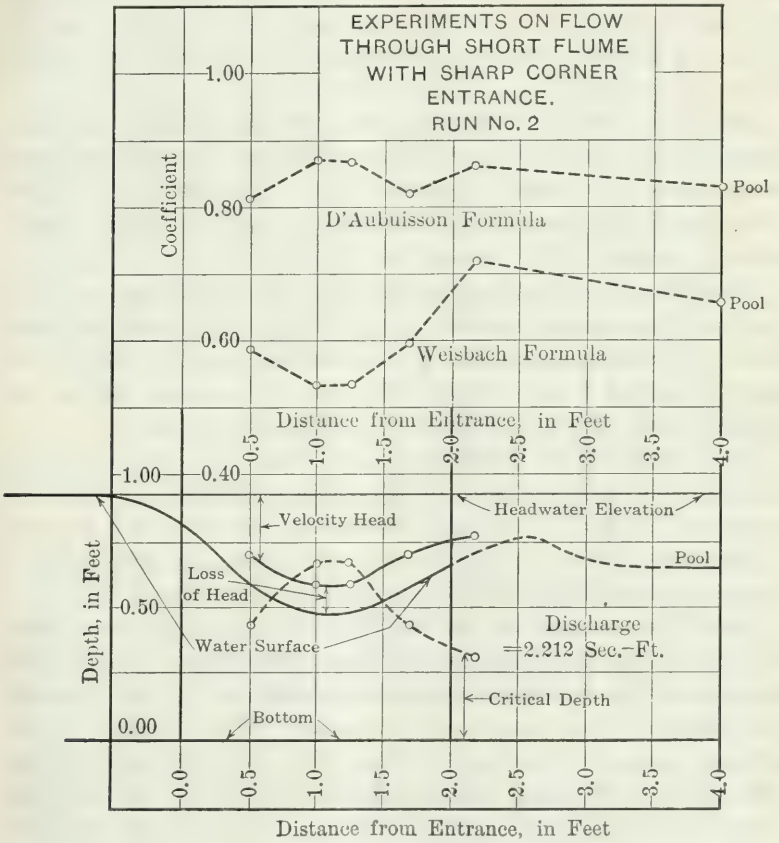


FIG. 15.

loss. If it was caused by friction, it would probably be greater in the narrow portion of the flume and would increase as the water passed down the flume.

Short Flume with Sharp Corners.—The results of the experiments on the short flume with sharp corners are given in Table 5 and shown on Figs. 14 and 15. The d'Aubuisson coefficients are fairly constant,

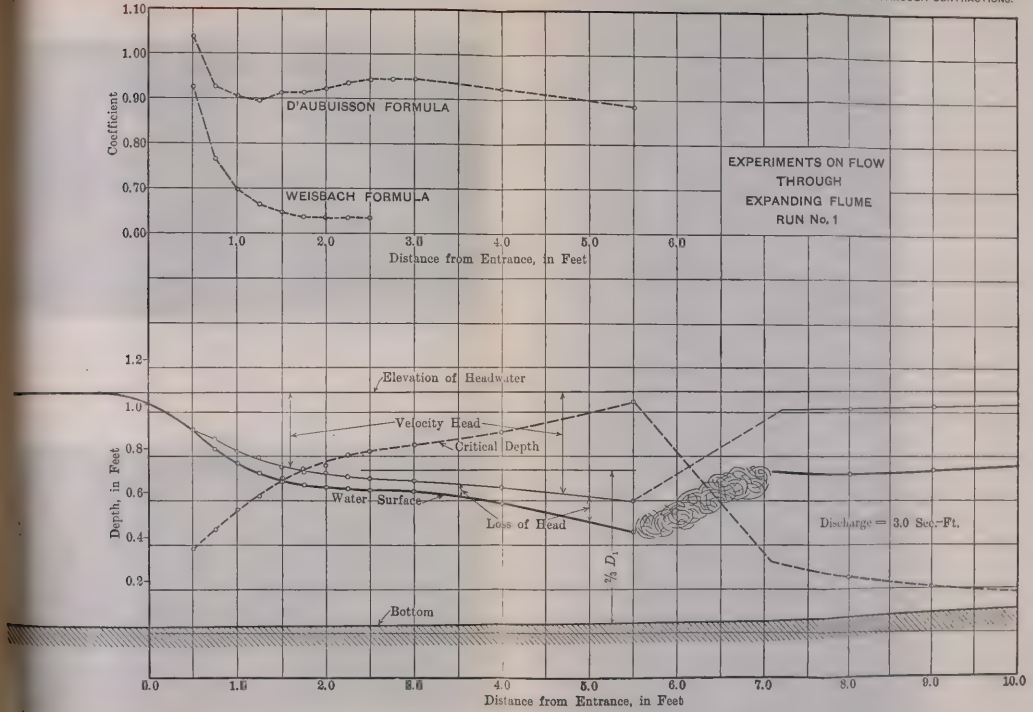
being between 0.80 and 0.90, while those of the Weisbach formula vary over a wide range. The d'Aubuisson coefficients are higher than one at first would expect, but it will be noticed that they are based on the actual area of flow, and, therefore, include the influence of the contraction resulting from the sharp corners only in so far as these introduce a somewhat turbulent state and thus cause internal friction and impact losses. This fact should always be kept in mind in estimating the flow through a contracted opening by the use of the d'Aubuisson formula.

The water surface in the flume fell below the critical depth for a stream having its velocity and, therefore, in order to reach the depth of the tail-water, formed a hydraulic jump. On account of the disturbed flow in this type of flume, the hydraulic jump was poorly defined, and did not take the form usually seen in such jump.

Expanding Flume.—The experiments on the expanding flume consisted of measurements under five conditions of head-water and tail-water, but all with approximately the same discharge. In view of the many interesting features of this flume, the writer regrets that he was unable to make a more complete series of experiments on it.

The results of the individual runs are shown on Tables 6 to 10, inclusive, and on Plates XIX to XXIII, inclusive, while a section of the flume, profiles of the water surface to natural scale, and the d'Aubuisson coefficients, for all runs, are shown on Plate XXIV. Views of two conditions of flow are shown on Figs. 16 and 17.

It will be seen that, for the first three runs, the d'Aubuisson coefficients in the prismatic section of the flume, are subject to less variation than those of the Weisbach formula, which indicates that the d'Aubuisson formula is the true one for this case. This coefficient in the prismatic section of the flume for Runs Nos. 1 to 3, varies little from 0.92, which was found to be the coefficient for similarly located points in the short flume with rounded entrance. As the expanding flume can be formed from the short flume with rounded entrance by adding the expanding portion, this agreement of the coefficients indicates that that portion has little influence on the flow through the narrow section of the flume. The high coefficient and negative loss of head at the 0.5-ft. point, and the great loss of head at the entrance, which occur in this flume, as in the short flume with rounded entrance, also point to this conclusion.


Coefficient
1.10
1.00
0.90
0.80
0.70
0.60
D'AUBUISSON FORMULA
WEISBACH FORMULA
EXPERIMENTS ON FLOW THROUGH EXPANDING FLUME RUN No. 1
1.0
2.0
3.0
4.0
5.0
6.0
Distance from Entrance, in Feet
1.2
1.0
0.8
0.6
0.4
0.2
Depth, in Feet
Elevation of Headwater
Velocity Head
Critical Depth
Water Surface
Loss of Head
Discharge = 3.0 Sec.-Ft.
½ D
Bottom
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
Distance from Entrance, in Feet

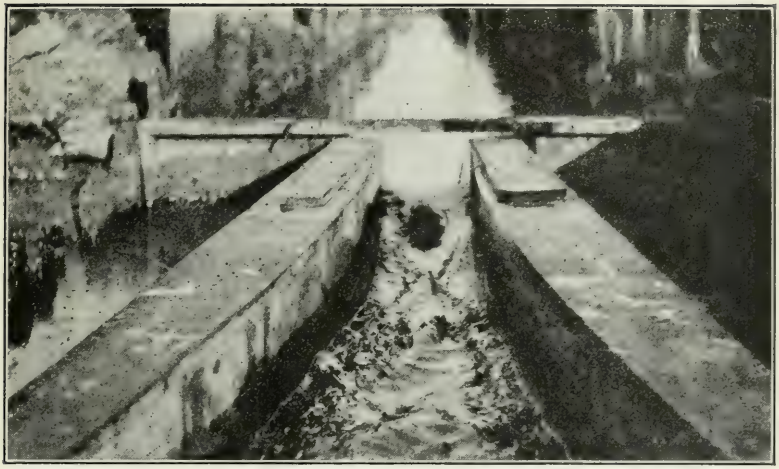


FIG. 16.—EXPANDING FLUME, RUN NO. 3.

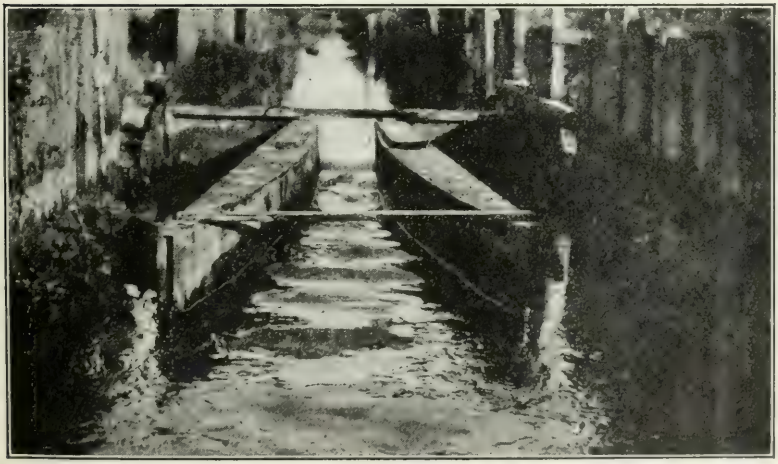
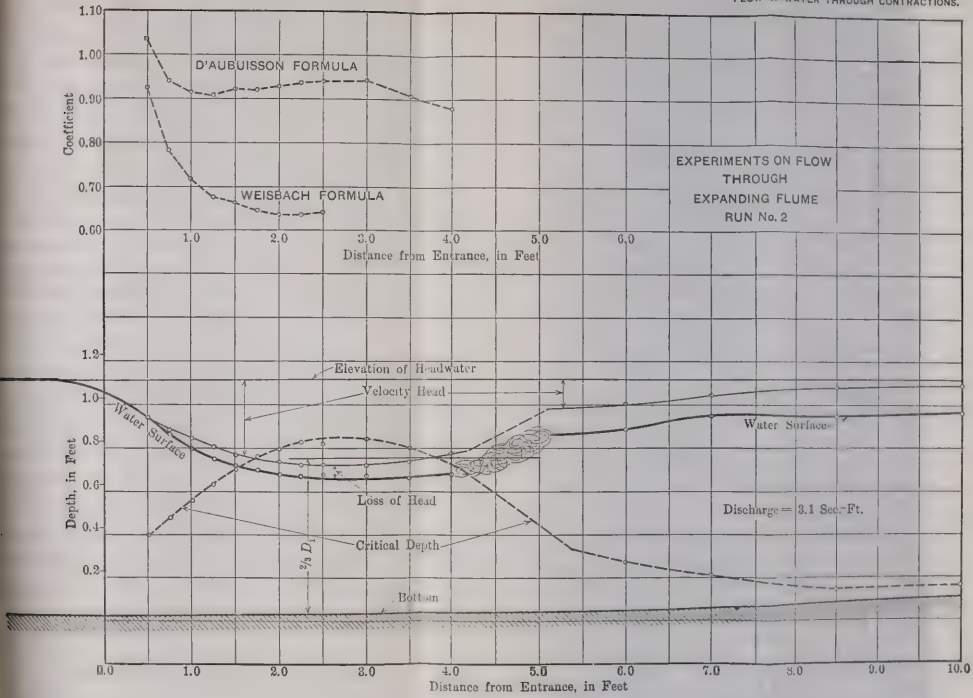


FIG. 17.—EXPANDING FLUME, SHOWING FORMATION OF STATIONARY WAVES.


D'AUBUISSON FORMULA
WEISBACH FORMULA
Coefficient
Distance from Entrance, in Feet
EXPERIMENTS ON FLOW THROUGH EXPANDING FLUME RUN No. 2
Elevation of Headwater
Velocity Head
Water Surface
Water Surface
Loss of Head
Discharge = 3.1 Sec.-Ft.
Critical Depth
⅔ D1
Bottom
Depth, in Feet
Distance from Entrance, in Feet

In Runs Nos. 4 and 5, and to a less extent in Run No. 3, the wave formation in the flume made the determination of the coefficients difficult. These waves were stationary, maintaining the same location in the flume, and smooth, resembling wind waves on a deep body of water. A further resemblance to wind waves was shown by the formation of a stationary whitecap in Run No. 4. This formation could easily be mistaken for the hydraulic jump, but as the velocity was far below the critical value for that depth, it resulted from another cause. As will be seen in Plate XXII, this rolling water was preceded by two steep waves, and was due, no doubt, to the breaking over of the crest of a wave, when it became too steep, as in the formation of whitecaps.

In Runs Nos. 1 and 2 the drop of the water surface exceeded the theoretical drop, $\frac{1}{3} D_1$, by an appreciable amount. This is probably due to the loss of head at the entrance. The results confirm the theory of maximum flow, however, in so far as they show that the water surface under such conditions drops to a certain depth, which is independent of the depth of the water below the flume.

One of the peculiar features of this flume is that the flow through it, within certain limits, is not affected by variations in the tail-water level. The variation in discharge, depth above, and depth below, the flume, during the period in which these experiments were made, are plotted on Fig. 18, which shows that, during the first three runs, the depth above the flume varies very little, while the depth below is changed through quite a range, by the introduction of stop-logs in the channel. What variation occurs during this part of the period of experimentation, may practically all be accounted for by the variation in the discharge through the flume. When a certain tail-water depth was reached, however, this relation ceased, and any increase in the depth below, the discharge remaining the same, caused an increase in depth above the flume. This condition existed during Runs Nos. 4 and 5. The cause of these conditions can best be explained as an application of the theory of back-water curves.

The Theory of Back-water Curves as Applied to the Expanding Flume.

S. M. Woodward, M. Am. Soc. C. E., in the "Theory of the Hydraulic Jump and Back-water Curves"*, has defined a back-

* Miami Conservancy District, Technical Report, Part III.

water curve as "the profile of the surface of a stream of water in which all changes of velocity take place in accordance with Bernouilli's theorem." In that report are developed the possible cases of back-water curves in a channel of uniform width, both with and without

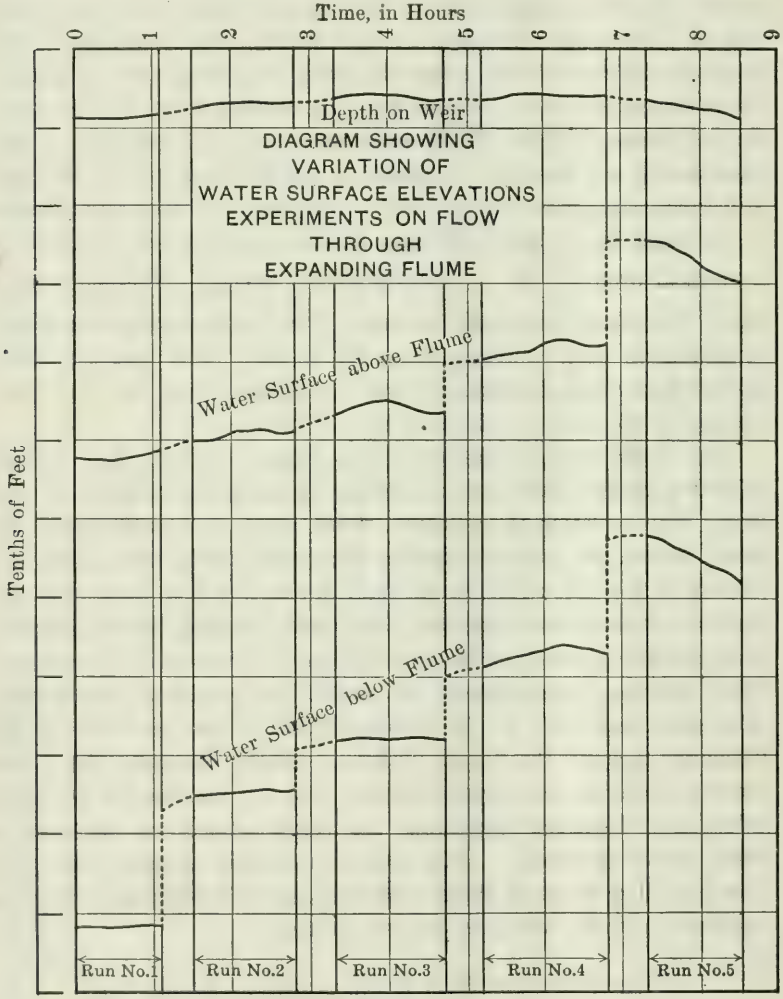


FIG. 18.

friction. As the friction loss in the expanding flume was relatively small, and as the theory of back-water curves, neglecting friction, is much simpler, the effect of friction will be neglected in this discussion.

PLATE XXI.
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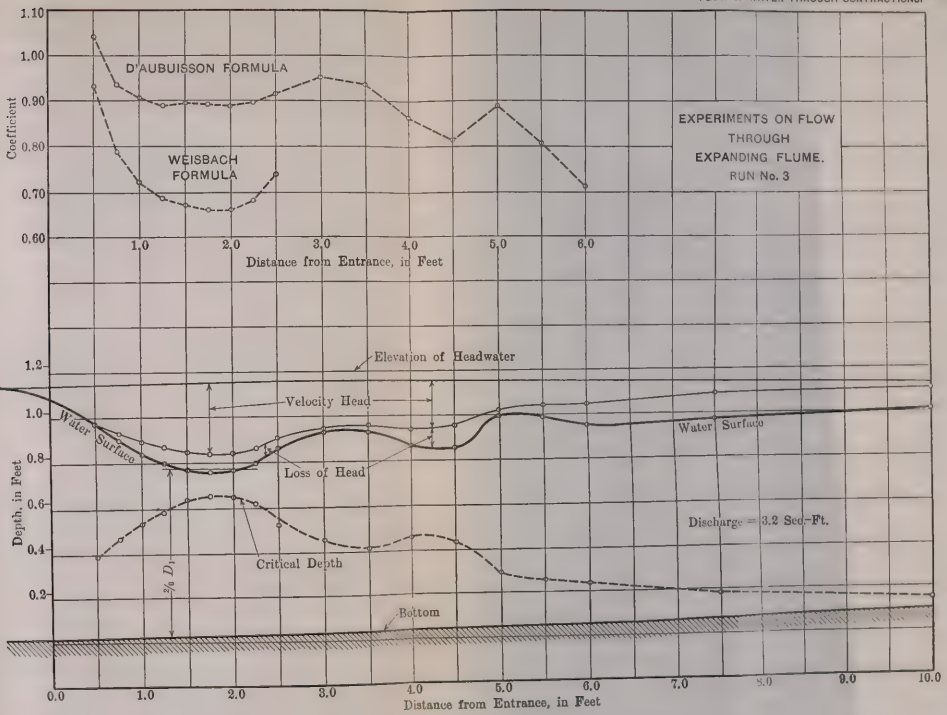
LANE ON
FLOW OF WATER THROUGH CONTRACTIONS.



According to Mr. Woodward, there are five possible conditions of flow in a frictionless channel of uniform width. They are:

Case 1.—Bottom level; water surface level; any depth.

Case 2.—Bottom slopes downward; water surface slopes downward at a greater rate than the bottom; depth less than the critical value.

Case 3.—Bottom slopes downward; water surface slopes upward; depth greater than the critical value.

Case 4.—Bottom slopes upward; water surface slopes downward; depth greater than the critical value.

Case 5.—Bottom slopes upward; water surface slopes upward at a greater rate than the bottom; depth less than the critical value.

The critical value of the depth in the foregoing is that depth at which $V = \sqrt{g\,D}$.

Using an illustration given by Mr. Woodward, a similar list of possible conditions of flow in a frictionless channel of variable width, with level bottom, may be worked out. Fig. 19 represents such a channel, in which the width is varied in such a manner that with a flow of 2 000 cu. ft. per sec. entering the channel at *A*, and flowing toward the right, the water surface rises along the straight line, *AB*. The width of this channel decreases rapidly near the entrance, but less rapidly as the water surface rises, finally reaching a point of minimum width, beyond which it again increases. Any other smooth curve might be used to join the points, *CDE* and *FGH*, but, in this case, the surface profile would no longer be a straight line, but a curve. Since for all points along the channel, the sum of the velocity and elevation heads remains the same, at each point on both channels having the same surface elevation, the width would be the same.

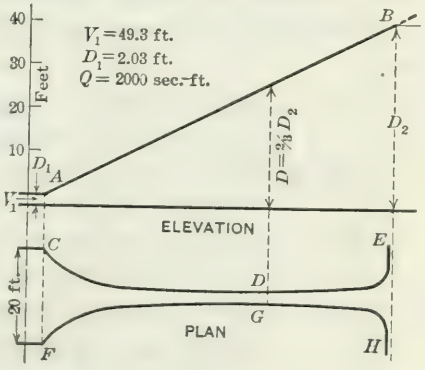


FIG. 19.

The point where the width is minimum can be found, as follows:

$$W = \frac{Q}{V\,D}$$

$$V = \sqrt{2\,g\,(D_2 - D)}$$

Differentiating, $$d\,W = d\left[\frac{Q}{\sqrt{2\,g\,(D_2 - D)}\,D^2}\right]$$

Equating to zero, and solving for D,

$$D = \frac{2}{3}\,D_2$$

From which, $$V = \sqrt{2\,g\left(\frac{3}{2}\,D - D\right)} = \sqrt{g\,D}$$

It will be seen, therefore, that the width is a minimum when the depth of the channel is at the critical value, as previously defined.

With the channel as shown in Fig. 19, the direction of flow might be reversed, the flow taking place from right to left. If the initial depth was D, and the quantity 2 000 cu. ft. per sec., as in the former case, the surface profile would be the same line, AB, the flow again being at the critical depths at the point of minimum width. In these two examples are shown the four possible conditions of flow in a frictionless channel with level bottom and variable width. They are:

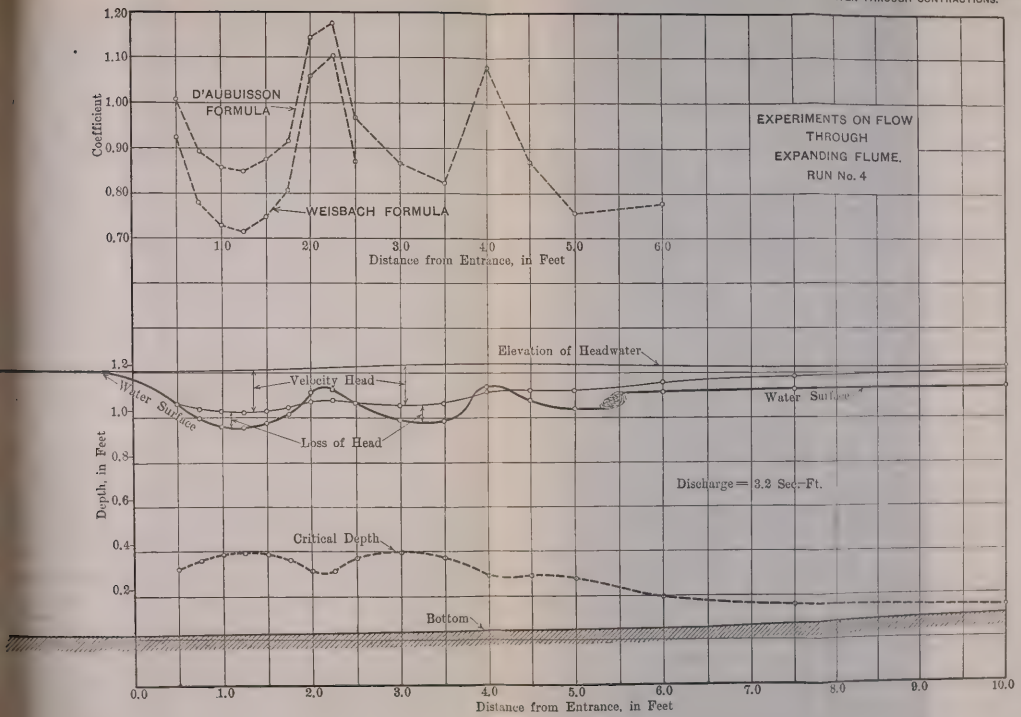
Case 1.—Width increases; water surface slopes upward; depth greater than the critical value.

Case 2.—Width increases; water surface slopes downward; depth less than the critical value.

Case 3.—Width decreases; water surface slopes upward; depth less than the critical value.

Case 4.—Width decreases; water surface slopes downward; depth greater than the critical value.

The foregoing conditions may be applied to the flow through the expanding flume. As the depth of the water below the flume is independent of the shape of the flume, we will start at this elevation. In all the experiments, this depth at the lower end of the flume was great, with respect to the velocity at that point, and was, therefore, above the critical value. On Plate XXV is shown the theoretical and actual surface profiles for Runs Nos. 1 and 4. The theoretical


Coefficient
D'AUBUISSON FORMULA
WEISBACH FORMULA
EXPERIMENTS ON FLOW THROUGH EXPANDING FLUME.
RUN No. 4
Distance from Entrance, in Feet
Elevation of Headwater
Velocity Head
Water Surface
Loss of Head
Water Surface
Depth, in Feet
Discharge = 3.2 Sec.-Ft.
Critical Depth
Bottom
Distance from Entrance, in Feet

profile has been worked out by the step method, on the assumption that the sum of the velocity and elevation heads at all points is the same, and that there is no friction loss.

In Run No. 1, according to Case 1, in an expanding channel, if the depth is above the critical value, the water surface must slope upward, as shown by the line, *AB*. This condition is possible, however, only when the flow is greater than the critical depth. In sloping upward in the direction of flow, and, therefore, downward from the tail-water level, this back-water curve soon reaches the critical depth. At this point, the curve ends, as it cannot go below that depth.

A method of locating the theoretical water surface above the flume is offered by the theory of maximum flow. According to this theory, the depth above may be computed by the formula, $Q = 3.09\ W_2\ D_1^{\frac{3}{2}}$. Knowing Q and W_2, D_1 may be computed. As the depth above the flume is greater than the critical value, from Case 4, it may be concluded that, in the entrance portion of the flume, where the width is decreasing, the water surface would slope downward. On reaching the prismatic section, from Case 1 for channels of uniform width, the water surface would be level. According to the theory of maximum flow, the depth in this section would be $D_2 = \frac{2}{3} D_1$, the velocity, $\sqrt{\frac{2}{3} g\ D_1} = \sqrt{g\ D_2}$, and, therefore, equal to the critical value.

Beginning at the lower end of the prismatic section, two curves are possible. If the water surface at this point drops an infinitesimal distance, the depth is below the critical value, and, therefore, from Case 2, the water surface must slope downward. (Line *CD*.) On the other hand, if the surface rises an infinitesimal distance, the flow takes place at greater than the critical depth, and, according to Case 1, the water surface must rise. (Line *CE*.) These two curves represent the "alternative stages" discussed by Karl R. Kennison, Assoc. M. Am. Soc. C. E.* Neither of these two profiles intersects the line, *A B*, and, therefore, if hydraulic phenomena were confined to applications of the Bernouilli theorem, the flow of the given quantity of water through the flume, with the given tail-water level, would be impossible. No phenomenon is known by which the water surface could change from the

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 347.

upper one of these curves and join the curve, *AB*, in a channel of the shape of the expanding flume. By means of the hydraulic jump, however, it is possible for the water surface to rise from the lower curve and join the curve, *AB*. The formula* for the hydraulic jump is:

$$D_2 = -\frac{D_1}{2} \pm \sqrt{\frac{2 V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

From this formula, it is evident that the jump can take place whenever $V > \sqrt{g D_1}$, or whenever the velocity exceeds the critical value. A jump might take place, therefore, at any point along the back-water curve, *CD*, as the flow is always at less than the critical depth. If a curve, *FGH*, is plotted above the curve, *CD*, at the height to which a hydraulic jump would take place, with the conditions of flow existing at that point, it will be found that this line intersects the curve, *AB*, at *G*. At this point, therefore, one would expect the water surface to rise in a hydraulic jump, from the line, *CD*, to the tail-water curve, *AB*.

The heavy line on Plate XXV shows the actual water surface for Run No. 1. Because of friction and impact losses, it starts at a greater elevation than the theoretical value. In entering the flume, it does not drop as rapidly as the theoretical curve, the reason for which has been explained previously. After the entrance water surface drop is complete, the profile follows closely the theoretical line, the jump taking place substantially as indicated by the theory, and the theoretical tail-water surface curve also agrees well with the actual surface profile.

A study of the theoretical back-water curves for Run No. 1 shows the reason for the different conditions of flow in the flumes, resulting from different tail-water levels. If the tail-water was at a lower elevation than the bottom of the flume, the flow in the expanding portion would follow the line *CD*, and drop off at *D* to the tail-water below. Until the water surface rose to the elevation *D*, the profile in the flume would be unchanged. As the water surface rose from *D* to *H*, it would still follow the line *CD*, and a phenomenon resembling the hydraulic jump would take place below the end of the flume. If the tail-water level is still further increased, the condition of flow in Run No. 1 would occur, the jump being formed in the flume

* This formula is usually accredited to Unwin, but according to Mr. S. M. Woodward, it was derived by a Frenchman, Belanger, in 1838.

PLATE XXIII.
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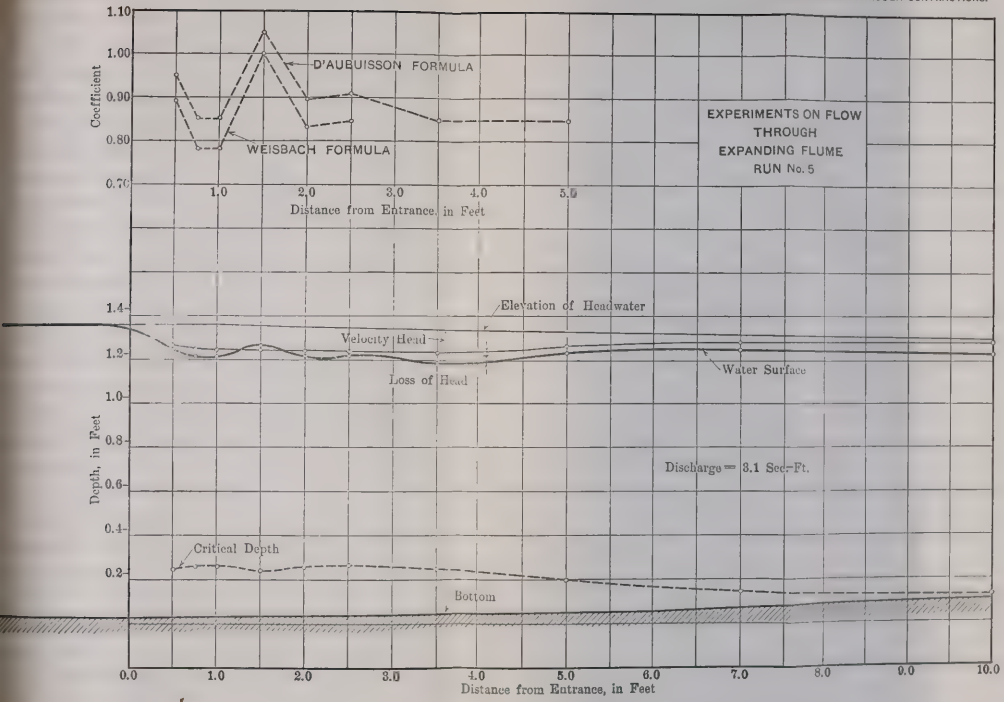
LANE ON
FLOW OF WATER THROUGH CONTRACTIONS.



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itself. Below the jump, the surface curve would slope upward, as *BGA*, and above the jump, the profile would follow the line *CD*. The jump would occur at the point where the back-water curve connecting with the tail-water intersects the line of height of jump, *FGH*. This condition of flow occurs for all tail-water heights between *H* and *E*, the jump moving up the flume as the water level below it is increased. Runs Nos. 1 and 2 are in this class. When the tail-water level reaches *E*, the curve connecting with the tail-water follows the line *CE*, and intersects the height of jump line at *C*, the end of the prismatic section of the flume. The depth here is at the critical value, and, therefore, no jump can take place. Run No. 3 closely represents this condition. For all heights of tail-water below, and including that at *E*, the flow through the entrance and prismatic sections of the flume remains the same, being the condition of the maximum flow. That this actually occurred is shown on Fig. 18, by the comparatively constant head-water elevation of the first three runs, the slight variation being practically all due to the slight change of discharge.

As the tail-water level rises above *E*, the back-water curve connecting with the tail-water falls above the line *CE*, and does not reach the depth of maximum flow. When the condition of maximum flow exists, the head-water elevation is the minimum which will discharge the given quantity. It follows, therefore, that if the depth in the prismatic section is greater than that of maximum flow, the elevation of the head-water for the given discharge must be greater than this minimum. As the tail-water level is increased above *E*, if the discharge remains the same, the head-water, therefore, rises. This is the condition of flow in Runs Nos. 4 and 5.

The theoretical profile for Run No. 4 is shown on Plate XXV. The back-water curve in the entrance section follows Case 4, which, for a channel of decreasing width, the depth of flow being above the critical value, gives a downward slope. In the prismatic section, the profile follows Case 1, of channels of uniform width, being horizontal. In the expanding section, the flow is according to Case 1, for channels of variable width, and the water surface slopes upward. A comparison of the actual with the theoretical profile for this run shows that, except for the effect of the waves, the actual water surface profile agrees very closely with the theoretical profile.

New Water Measuring Devices.

Although the experiments described in this paper were not undertaken primarily for the purpose of developing apparatus for the measurement of flowing water, some of the devices experimented on possess considerable promise as measuring devices. This is particularly true of the sharp edge contractions and the expanding flume. Although the contraction with rounded entrance might be used for water measurement, all of its advantages and none of its disadvantages are found in the sharp edge contraction. The same is true with respect to a comparison of the short flume with rounded entrance and the expanding flume, the latter possessing all the advantages of the former, and other good points in addition. The few experiments on the short flume with sharp corners do not indicate that it has any particular value for measuring water.

The requirements of the ideal measuring device for irrigation purposes have been stated by Mr. V. M. Cone,* as follows:

"The ideal measuring device would (1) be inexpensive to construct, (2) be simple to operate, (3) require little maintenance, (4) be free from working parts, (5) be accurate in its measurement, (6) be free from sand, silt, or floating-trash troubles, and (7) require but little loss of head in the ditch."

The sharp edge contraction seems to fulfill all these requirements. It combines the advantages of the sharp edge weir and submerged orifice, without the disadvantages of either. A somewhat similar device has been developed by Mr. Cone,† but it requires greater fall in the ditch, as the water surface below the "weir" must be below the bottom of the approach channel.

To adapt the expanding flume to the measurement of water, it would be necessary, in addition to a gauge box in the channel above the flume, to connect a still box into the side of the flume about 2 ft. below the upper end, in order to read the depth of water in the throat. Although in most cases it would be necessary to read only the up-stream gauge, where conditions are such that no hydraulic jump is formed it would be necessary to read both gauges. A much simpler shape of flume might be devised, using the same principle as the writer's flume, but better adapted to working conditions. A flume of this sort has

* "The Venturi Flume", *Journal of Agricultural Research*, Vol. IX, No. 4.
† "A New Irrigation Weir", *Journal of Agricultural Research*, Vol. V, No. 24.

PLATE XXIV.
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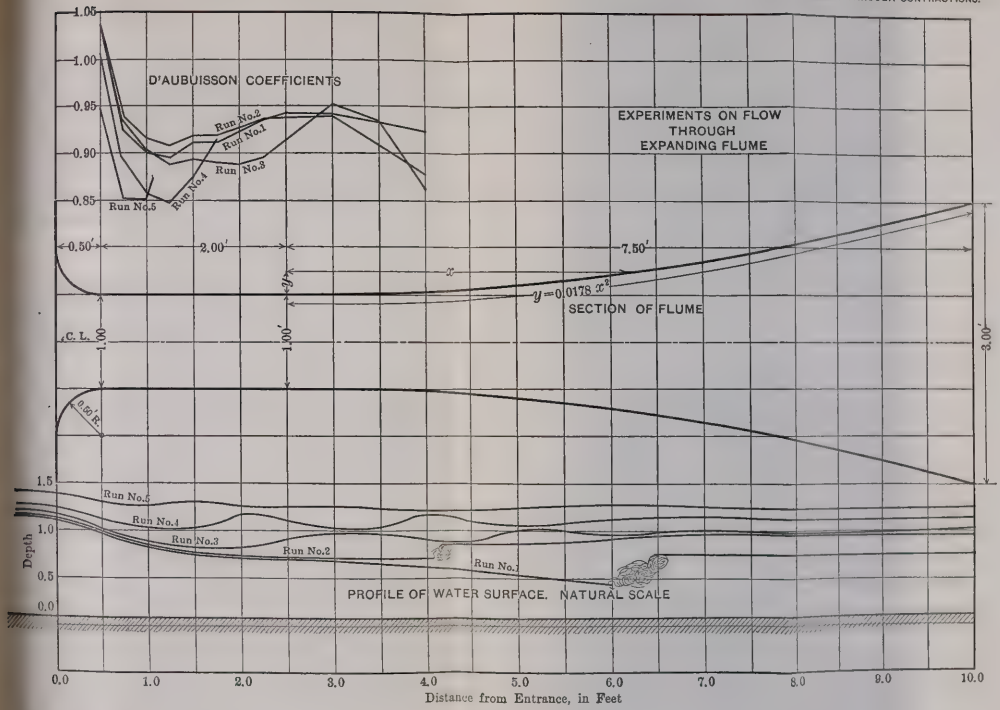


been worked out independently by Mr. Cone, and described in his article on "The Venturi Flume", previously referred to. The experiments on this flume, however, covered only those stages where $D_2 > \frac{2}{3} D_1$. In limiting its use to this range, the writer believes that one of the important advantages is overlooked, namely, that the discharge at most stages is independent of the tail-water level. This feature makes the action of the expanding flume similar to that of the weir, while, at the same time, it is free from the silt troubles of a weir and requires much less loss of head. The writer very much regrets that he was prevented from carrying out a thorough test of this device, and would be glad to see it taken up by some one having the proper facilities, and developed into a practical measuring apparatus.

Conclusions.

In conclusion, the writer wishes to reiterate the common, but often neglected, warning of using formulas blindly in cases to which they do not apply, or beyond the limit of the conditions for which they were developed. This has been expressed by Mr. Woodward,* in the following words:

"Perhaps the most frequent error in hydraulic discussions, in general, is the very common one of applying a law, rule, or formula to a situation where it does not hold. Every rule in hydraulics is limited in its application to a definite set of conditions outside of which it may have no bearing whatsoever. This tendency to apply a perfectly good rule or formula to the wrong situation is naturally not only difficult to avoid in the first instance, but also not easy to refute by argument after the mistake has been made."

The number and diversity of the formulas considered in this discussion are indicative of the many conditions under which water may flow through a contraction in a channel. It is obvious that the application of any of them to all conditions, will lead to very erroneous results.

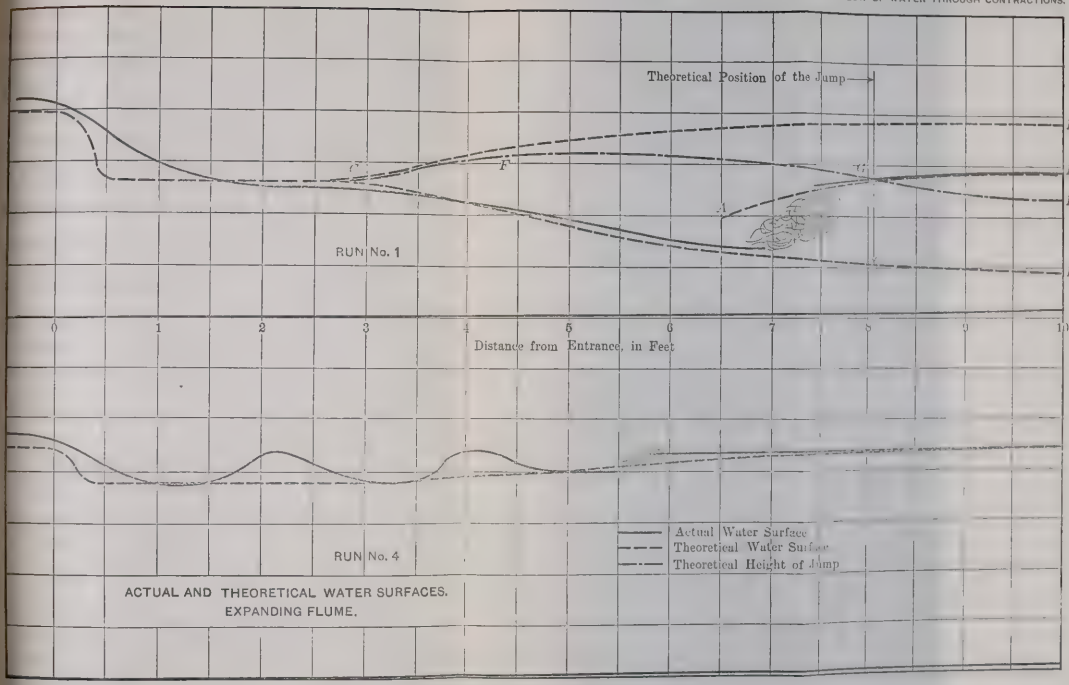
The d'Aubuisson formula is really an application of the Bernouilli theorem, with a coefficient to take care of the losses. The conditions in which it can be used are, in general, the same as those of the Bernouilli theorem. It can best be applied to contractions in which

* "Theory of the Hydraulic Jump and Back-water Curves," Miami Conservancy District, Technical Report, Part III, p. 11.

the water flows from one stage to another without a great deal of turbulence. The two areas of measurement must be the actual area of flow of the water, and the conditions at these points such that the static pressure on each particle is equal to the weight of the water above it, up to the free water surface. These conditions are fulfilled by the flow through the prismatic sections of the expanding flume and the short flume with rounded entrance, and also in the contraction with rounded entrance, where $H_3 < \frac{1}{3} D_1$. To a certain extent it can be applied to the short flume with sharp corners, the turbulence introduced by the sharp edges causing a lower value of the coefficient. It can rarely be applied where a reduction of velocity occurs, unless this reduction is gradual, as in several of the experiments on the expanding flume.

The conditions most favorable to the Weisbach theory are those existing at a sharp edge contraction of narrow width. The drop of the water surface above or in the contraction must be as small as possible, and the stream, after passing the contraction, must be free to expand laterally, since it is necessary that no pressure is exerted on those particles passing through the contraction above the tail-water level, in order that they may acquire a velocity corresponding to their distance below the head-water level. If the sharp edge contraction is wide, there will be a greater drop in the water surface above the contraction, and the formula will be less exact in its application. As the condition of flow in the case of a sharp edge contraction is very similar to that through an orifice or over a weir, the coefficients of both should be approximately the same.

On account of the surface drop which always takes place above a contraction, the condition assumed in the Weisbach theory will be only approximately attained. By modifying the Weisbach formula by giving different coefficients to that part of the flow above tail-water level, and that below it, a formula can be obtained, which will give results with a high degree of accuracy. The difference in the coefficients for these two parts is caused by the surface drop above the contraction, which reduces the flow through that portion above the tail-water level only. This formula is probably applicable to a great range of widths of the sharp edge contraction, and one may expect the coefficient, for that part of the flow below the tail-water level, to increase, while that for



ACTUAL AND THEORETICAL WATER SURFACES.
EXPANDING FLUME.

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the part above the tail-water level decreases, as the width of the contraction is increased.

The general formula for contractions (Formula (18)) is adapted to contractions of short length, in which the stream is free to expand immediately after passing the narrowest point. This formula, in special cases, may take the form of either the d'Aubuisson or the Weisbach formula. The coefficients in it are substantially the same as those of an orifice of the same character of edges. This formula has the disadvantage that it contains the term, H_2, the drop of the water surface at the most contracted section, which is often difficult to measure.

Formula (12) is very similar to the general formula, but the term, H_2, of the general formula is replaced by the term, $\frac{1}{3} D_1$. For the contraction with rounded edges, where $D_3 < \frac{2}{3} D_1$, to which Formula (12) is applicable, H_2 is substantially equal to $\frac{1}{3} D_1$. For the contractions, the coefficients have been found to be a function of $\frac{H_3}{D_3}$.

The flow in the expanding flume follows very closely the theory of back-water curves. For a frictionless channel, with level bottom and variable width, the following four back-water curves are found to be possible:

Case 1.—Width increases; water surface slopes upward; depth greater than the critical value.

Case 2.—Width increases; water surface slopes downward; depth less than the critical value.

Case 3.—Width decreases; water surface slopes upward; depth less than the critical value.

Case 4.—Width decreases; water surface slopes downward; depth greater than the critical value.

A stream is flowing at the critical depth when $V = \sqrt{g D}$.

The condition of "maximum flow" has been found to exist substantially as indicated by the theory. The flow may exceed this "maximum", however, where the stream is allowed to expand at or immediately after the point at which the depth of maximum flow is reached.

The sharp edge contraction and the expanding flume are believed to possess value as measuring devices, and should be investigated further.

From Appendix II, it may be concluded that when a channel, in which water is flowing at high velocity, changes abruptly into a wider channel, with a bottom approximately level and at nearly the same elevation as that of the narrower channel, an unstable condition of flow may result.

ACKNOWLEDGMENTS.

The writer wishes to express his appreciation of the assistance of the following: To A. E. Morgan, and C. A. Bock, Members, Am. Soc. C. E., for encouragement in undertaking these experiments; to the Morgan Engineering Company and the Miami Conservancy District for the use of instruments, without which the experiments could not have been performed; to S. M. Woodward, and R. M. Riegel, Members, Am. Soc. C. E., Ivan E. Houk, Assoc. M. Am. Soc. C. E., and Mr. J. D. Burrage, for valuable suggestions and for assistance in preparing the manuscript; and to many of the Engineering Staff of the Miami Conservancy District, who, from time to time, assisted in the experiments.

APPENDIX I.

Application of the Theory of Flow Through Contractions to the Design of the Flood-Prevention Works of the Miami Conservancy District.

In the investigation for the design of the flood-prevention works of the Miami Conservancy District, one of the first problems encountered was that of the flow of water through a contraction in a channel. Soon after the flood had subsided, an extensive hydrographic survey was made to determine the discharge and slope of the river, and the area covered by the flood at its highest stage. It was found that a considerable drop in the water surface occurred at a number of bridges. The magnitude of the drop could be determined from the high-water marks and the area of the stream above, and, in the narrow section, could be easily measured. If the law relating to these areas, the drop, and the discharge of the stream, were known, a convenient method of determining the flood discharge was available.

The method of computing the peak flow from these measurements has been given in detail elsewhere,* and, therefore, will not be described here. The method used was devised by Messrs. S. M. Woodward and Ivan E. Houk. Essentially, it is the d'Aubuisson theory with a correction based on a computed friction loss. As the Weisbach formula was the nearly generally used form, these experiments were originally undertaken to compare the merits of the two theories. The results of these experiments show that, for the conditions at these bridge openings, the method used was much superior to the Weisbach formula.

Another case involving a determination of the drop which takes place at a contraction, arose in connection with the design of the retarding basins which are to be used in this flood-prevention system. Five large dams are being built across the Miami River and its tributaries to form basins for the storage of flood-water. Through these dams will be conduits, through which the ordinary discharge of the river will pass, but which are not large enough to discharge the flood flow of the river. The flow in excess of the capacity of these conduits will be retained in the basin above the dam until the flood peak is

* "The Official Plan of the Miami Conservancy District," Vol. I, pp. 67-71; Technical Report of the Miami Conservancy District, "Calculation of Flow in Open Channels," Part IV.

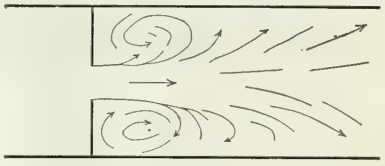
passed, and will then be discharged through the conduits into the stream again. In several of the dams, these conduits consist of large openings through a short spillway, which is a section of ogee dam, flanked at the ends by high concrete retaining walls. In order to take care of the flow of the stream during the construction period, it is proposed to build these walls first, leaving out the ogee spillway and conduit section until the earth section of the dam is complete. In times of freshet, all the water must pass through this comparatively narrow opening, and, in times of flood, considerable heading up will take place. As these dams are above populous cities, the backing up of the water to such an extent as to overtop the partly completed fill would be disastrous. In order to estimate accurately the height to which the water would rise behind the dam during a flood, a knowledge of the true laws governing the flow of water through a contracted opening was necessary.

To estimate the area and frequency of the flooding of land which will take place in the future, as a result of the construction of the detention dams, computations were made to ascertain the area which would have been covered by the floods that have occurred in the past. In several of these dams, the conduits are large, and as the floor of the basin is very flat, considerable land would be flooded in the basins before the water reached the top of these openings. In many of the smaller floods, the flow would never be sufficient to reach the top of the conduits. While the basins were filling to this point, the flow would take place through the conduits in the same manner as through a bridge in which the piers occupy a large portion of the waterway. Here, again, in order to determine the backing up which would take place, an understanding of this phenomenon of flow was necessary.

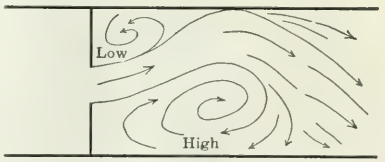
APPENDIX II.

Unstable State of the Stream from the Contractions.

Whenever a channel in which water is flowing at high velocity changes abruptly into a wider channel, with a bottom approximately level, and at nearly the same elevation as that of the narrower channel, an unstable state of flow may occur. The conditions under which this took place in the experiments on the contractions are shown in Fig. 20. When the water started to flow through the contractions, it passed down the center of the channel below, with a slight eddy on either side. (Fig. 20 (*A*)). This was an unstable condition, however, and soon the stream began to waver slightly and then was deflected to the side, as shown on Fig. 20 (*B*). This was a stable state, and the flow would continue in this position indefinitely. It was quite difficult to deflect it back into a central position again. At first, the writer thought that it was due to slight irregularities in the bottom, but as the deflection seemed to take place as often to one side as to the other, and as he had observed the same phenomenon in another apparatus under somewhat similar conditions, the conclusion was reached that this was not the cause.



(*A*)



(*B*)

Fig. 20.

When the stream from the contraction flowed in the center of the channel, the friction of this moving water on the quiescent water on both sides tended to draw this quiet water away from the contraction, making the tail-water level near the contraction lower than at some distance down stream. The water level on both sides of the flowing stream in this unstable state, was the same. The higher water down stream, however, tended to flow back toward the contraction, and if a slightly greater quantity flowed back on one side of the stream than on the other, the water level on that side became higher. The quiet water on this side, therefore, exerted a greater pressure on the flowing stream than the water on the opposite side, tending to force the flowing stream

toward the lower side. This allowed a larger quantity of water to flow back on the higher side, causing a still greater deflection, which increased until the position shown on Fig. 20 (*B*) was reached. The change of the flowing stream from a central position to a permanent deflection to the side of the channel was a matter of only a few seconds.

To remedy this condition, a low submerged weir was placed across the channel, 8 ft. below the bulkhead (Plate XVIII), which introduced a slight drop in the water surface at this point. The water surface below this weir was lower than the surface of the tail-water on the two sides of the stream, near the contraction, and there was no tendency of the water, therefore, to flow up the channel toward the contraction and cause a deflection of the stream.

DISCUSSION

Mr. Lyle.

WILLIAM T. LYLE,* ASSOC. M. AM. SOC. C. E. (by letter).—The conclusions drawn from these carefully prepared experiments are characterized by the cogency of their logic. The author regrets the fact that owing to the war his examination was not carried further. It is to be hoped that so profitable an undertaking may be continued, not only in the interest of hydraulic science, but also for the practical benefit of the profession. The author very properly calls attention to the folly of using hydraulic formulas beyond the limit of their application. A formula must be modified or changed whenever a new condition arises in the flow. The danger here involved has been forcibly brought home in the use of run-off formulas; but there are many other examples as well, and many a hidden rock on which professional reputations have been damaged.

Any device for contracting the cross-section of a jet or stream of water constitutes a measuring device. Thus, there are orifices, weirs, Venturi meters and contracted open channels. The underlying principle in them all is the hydraulic expression of the law of the conservation of energy, namely, Bernouilli's theorem—pressure head plus velocity head plus friction head is a constant.

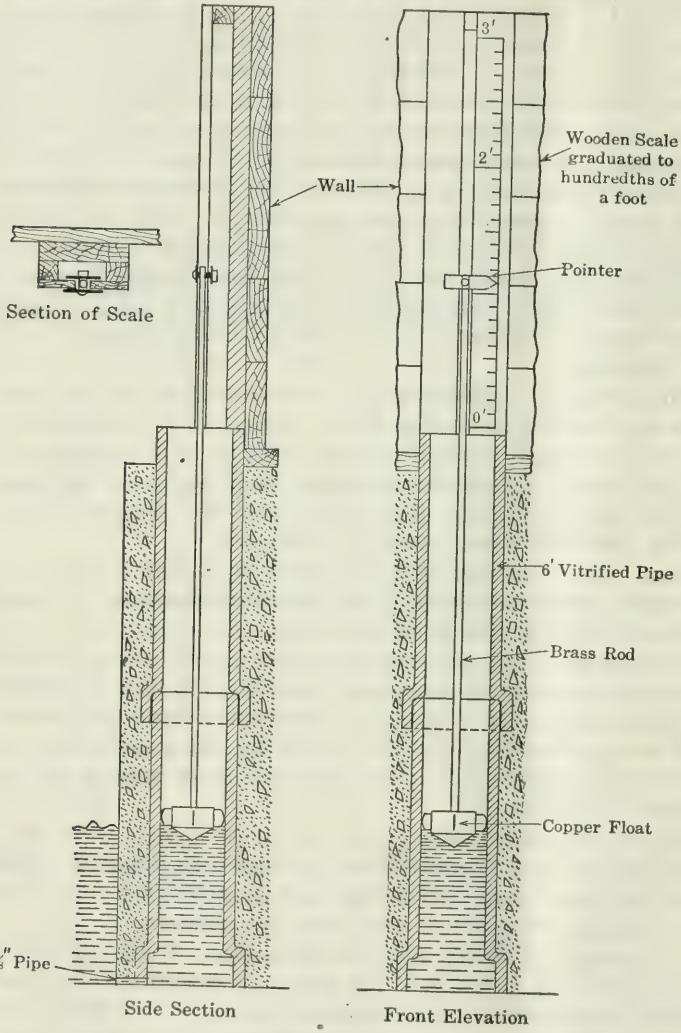
Some years ago, the writer designed a stilling box for use in volumetric tank measurements and weir determinations. It consisted of a copper float, convex on the lower side, with a brass stem to hold an index which passed up and down through a slot on the recording scale. All graduations were on wood which is believed to be a small improvement on the steel tape referred to in the paper because of the fact that wood does not corrode. The float moved up and down through a riser of sewer pipe, with a small hole for the entrance of water at the bottom, as shown in Fig. 21.

In order to guard against any possibility of binding, it was the practice to tap the index lightly with the finger before taking a reading. This float gauge gave good results and is better than a hook gauge, where rapidity and elimination of surge are necessary.

In discussing Formula (4), the author says: "Part (*b*) of the discharge, according to the Weisbach theory, is exactly equal to the total discharge according to the d'Aubuisson theory." This is true providing the coefficients are the same. The separate coefficients appear in Formula (14). The writer questions the strict applicability of Formula (12), which applies to orifice flow. This same equation appears in modified form again as Formula (18). The distinction made between stream line and turbulent flow is not well taken. The stream-line theory is now obsolescent, if not obsolete.

* Houston, Tex.

Mr. Lyle. The author very properly calls attention to the great advantage of the expanding flume over the weir for water measurement, the flume



FLOAT GAUGE

FIG. 21.

being free from clogging by floating matter, free from silt deposits, and requiring much less loss of head.

Mr. Henny. D. C. HENNY,* M. AM. SOC. C. E. (by letter).—Under the head of "New Water Measuring Devices", the author refers to the Venturi

* Portland, Ore.

flume suggested by Mr. V. M. Cone.* A somewhat different and equally simple device using the Venturi principle was suggested by the writer in 1915, for the purpose of measuring flow in irrigation laterals. It consists of an ordinary rating flume with parallel sides, the only addition being a thin, galvanized steel plate fastened to the sides, which depresses the water surface so as to contract the flow area to a throat, as shown by Fig. 22. Mr. Henny.

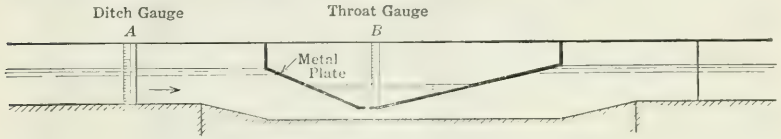


FIG. 22.

For ordinary irrigation heads of from 3 to 6 sec-ft., the throat is 4 in. high and 2 ft. wide, the throat velocity varying from 4.5 to 9 ft., with corresponding velocity heads of 0.31 to 1.26 ft. Two vertical staff

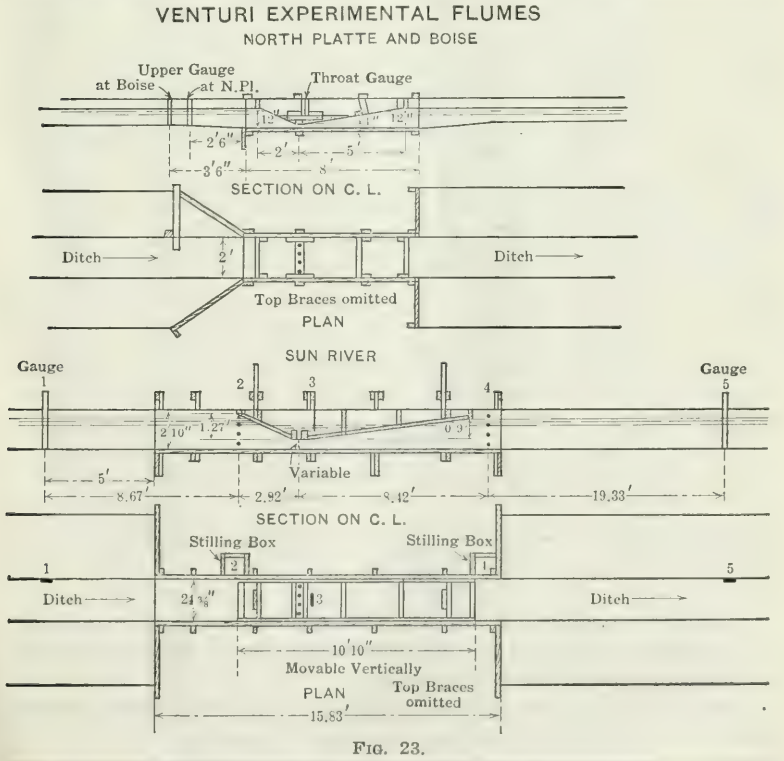


FIG. 23.

* *Journal of Agricultural Research*, Vol. IX, No. 4, April 23d, 1917.

Mr. Henny.

gauges, A and B, are read, the upper one placed in the ditch above the wings of the flume, the lower one in the head-water basin above the metal plate which communicates through small holes in the throat with the flowing water.

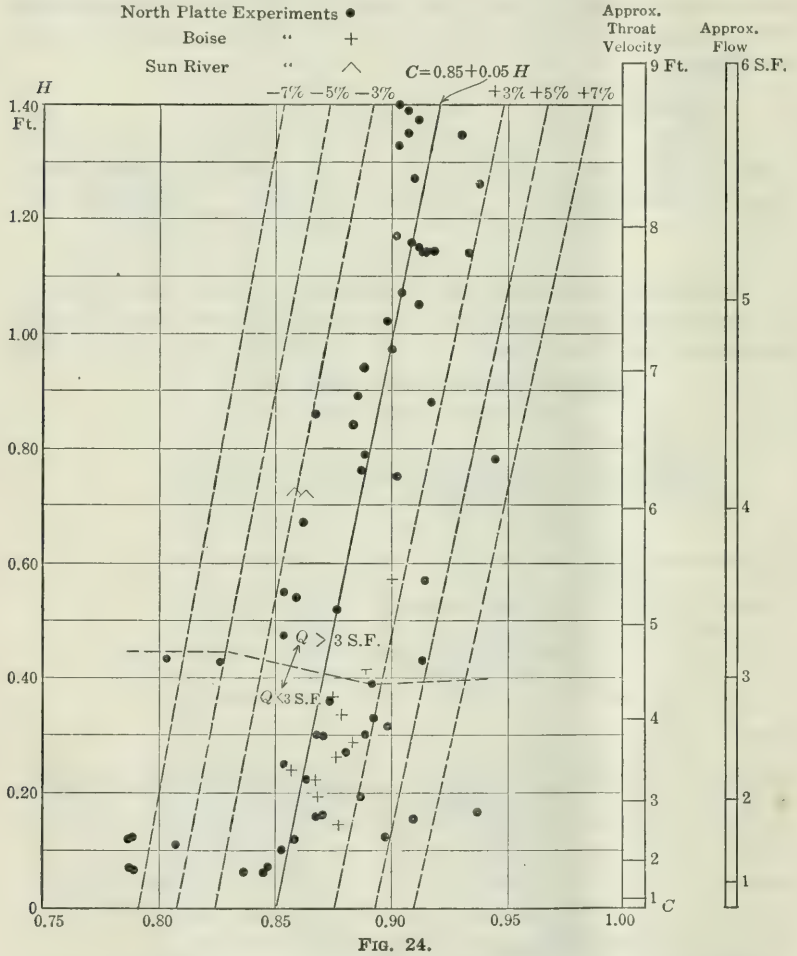


FIG. 24.

For a flow varying from 3 to 6 ft., the velocity in the approach ditch may range from 1 to 1.5 ft. Allowance is made in the formula used for the mean velocity head of approach.

Experiments made on three reclamation projects with flumes similar to that shown in Fig. 23, indicate that, in the formula,

$$V \text{ (Throat Velocity)} = C \sqrt{2\,g\,(H + h)},$$

in which H is the difference in gauge readings and h the mean velocity head of approach, the value of C increased with H. For the dimensions of flumes used in the experiments, the formula becomes: Mr. Henny.

$$V = (0.85 + 0.05\,H)\,\sqrt{2\,g\,(H + h)}$$

The flow thus determined in 39 experiments in which the throat velocity was 3 ft., or more, was compared with the flow measured by standard Cipolletti weirs with the following results (see Fig. 24):

35	measurements	show	error	of	less	than	3	per	cent.
3	"	"	"	"	"	"	5	"	"
1	"	"	"	"	"	"	7	"	"

These results appear to be quite satisfactory, considering that the weirs themselves are subject to error and that they represent three devices built and tested at widely separated points and by different experimenters.

The device is cheap, and the gauges can be easily read by an ordinary ditch rider. There is little danger of silting and the loss of head where flaring wings are used, both down stream and up stream, can be held down to less than 20% of the velocity head at the throat, or for the dimensions and maximum flow in question, to less than 3 in. The accuracy is not seriously affected by the usual range in variation of back-water.

Under certain conditions this form of Venturi flume is likely to be more accurate than the weir, partly because a small error in gauge reading has a smaller effect. It was intended, however, especially for cases where the available head is small or where the water carries considerable silt.

KARL R. KENNISON,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read Mr. Lane's valuable contribution to the literature on the subject of high-velocity flow in open channels, and has been particularly interested in the experiments in which the channel was given a smooth rounded entrance, producing a fairly smooth stable flow through it, with the internal pressure conditions in such channel not upset by a sharp-cornered entrance. Mr. Kennison.

One point which the writer wishes to make in this connection is that the quantity of discharge through a channel or conduit of this type may be governed by other conditions than the depth of water down stream, that each condition of flow must be studied by itself to determine just what it is that controls the quantity of flow, and that it is usually a simple matter to make this computation with remarkable precision, even when high-velocity flow and the hydraulic jump are involved.

* Boston, Mass.

Mr. Kennison.

The conclusion to be drawn from the experiments is a justification of the d'Aubuisson formula, in other words, the Bernouilli theorem. Everything in the experiments confirms this theorem and, consequently, justifies the computation of maximum discharge as the discharge at a depth equal to two-thirds the total net head, or twice the velocity head. Accordingly, if the down-stream head on the controlling section is less than two-thirds the total net head, it is too low to have any effect on the quantity discharged (through a channel or conduit long enough to contain a reasonably uniform flow). It is also true that the down-stream head can be quite a little greater than this two-thirds depth without materially decreasing the discharge. This characteristic of the water surface near the two-thirds depth has a tendency to create surface waves.

TABLE 11.—FLUMES WITH ROUNDED ENTRANCE.

Channel, 1 Foot Wide; Radius of Entrance Sides, 6 Inches.

SEE AUTHOR'S TABLES 4, 6, 7, 8, 9, 10.	SHORT FLUME.			EXPANDING FLUME.				
Run number.	1*	2*	3	1	2	3	4	5
Up-stream head, in feet	1.41	1.31	1.16	1.05	1.08	1.11	1.18	1.30
Drop to pool	0.608†	0.453	0.362	0.33	0.159	0.119	0.093	0.067
Down-stream water level	0.802	0.857	0.798	0.72	0.921	0.991	1.087	1.233
Effective down-stream head‡	0.802	*0.857*	*0.798*	‖	*0.68*	*0.82*	*0.96*	*1.14*
Loss at entrance§	0.084	0.072	0.058	0.062	0.064	0.05	0.04	0.03
Net up-stream head	1.326	1.238	1.102	0.988	1.016	1.06	1.14	1.27
Two-thirds of up-stream head¶	*0.884*	0.825	0.735	*0.659*	*0.678*	0.71	0.76	0.85
Velocity head	0.442	0.381	0.304	0.329	0.338	0.24	0.18	0.13
Computed discharge, in cubic feet per second	4.72	4.25	3.53	3.03	3.17	3.2	3.3	3.3
Measured discharge	4.735	4.267	3.537	3.0	3.12	3.2	3.2	3.1

* The numbers of these runs appear to be interchanged in Table 4.

† 0.508 in Table 4 is apparently an error.

‡ Before velocity head is recovered (with, say, 20% loss) in expansion to 3 ft. width.

‖ Too low possibly to affect the discharge; hydraulic jump must occur.

§ Velocity head in flume, 19%, equivalent to discharge coefficient of a little less than 0.92.

¶ Use this depth if greater than effective down-stream head.

One point which should be borne in mind is that the undulating surface profile does not represent the pressure head on the water beneath the surface, as one might be led to expect from the author's computations and plotted results. The vertical height based on this undulating surface which he has marked "loss of head", and which appears to increase and decrease through the length of the channel, is not the actual loss of head. The actual loss continually increases and can never be recovered. The author's profile would have been clearer, if this loss of head had been plotted vertically down from the "elevation

of head water", in such a way as to show the net available head continually decreasing. Then the critical two-thirds depth instead of being plotted as two-thirds of the total original head, should have been plotted as two-thirds of the net head after subtracting this loss. This then would have shown a more remarkable coincidence between the actual depth in the channel and the theoretical two-thirds depth for a maximum unobstructed flow. Mr. Kennison.

To bring out this point more clearly, the writer has taken all those experiments in which the rounded entrance was used, has used a constant coefficient for this entrance of a little less than 0.92, which is the coefficient apparently indicated by the experiments, and has computed the discharge by analyzing each case separately, both with and without the expanding flume, disregarding entirely the down-stream water level when it is lower than the critical two-thirds depth. The computed results (Table 11), compare remarkably well with the measured discharge, undoubtedly well within the precision of the measurement of the discharge.

R. D. Goodrich,* M. Am. Soc. C. E. (by letter).—This paper is believed to be a most valuable contribution to the literature of hydraulic investigations and is certainly one of great interest to all engineers dealing with problems of river training and regulation, as well as to those interested in irrigation and power development. Mr. Goodrich.

The complete description of the plant and methods used, with the illustrations, give confidence in the accuracy of the tabulated data and the results shown. The Cippoletti weir is not usually considered to be as accurate a measuring device as the rectangular weir with suppressed contractions. However, in this case, where special experimental coefficients are used, there is little doubt as to the reliability of the measurements. The author's descriptions of, and comments on, the float gauges are of considerable importance, and it would also be interesting to know what led to the selection of the particular curve used for the sides of the expanding flume.

The experiments show conclusively that the coefficients in the two formulas investigated are far from constant. Although these coefficients vary with the nature of the entrance to the contraction, it is clearly shown that this variation is some function of the effective head and depth of the contraction and also in one case of the width. It is possible that the coefficients for rounded or other forms of entrance vary with the width, as well as those for the sharp edged contractions. The author's results on the two different widths of the sharp crested weir agree in this regard with the coefficients recommended by Mr. W. G. Beigh from his observations and measurements made in India, which approach unity as the width of the contraction increases.

* Tientsin, China.

Mr. Goodrich.

Although the author has treated this variation as linear and although Fig. 10 would indicate a straight line law for the sharp edged contraction, it does not follow that it would be so in all cases; C might be expressed by a relation of the form:

$$C = a + b\left(\frac{H_3}{D_3}\right)^n$$

in which the law becomes linear when $n = 1$, as for the sharp edged contraction. The writer found that with $n = 2$, a single equation could be found to express the probable variation in the d'Aubuisson coefficient for the round edged contraction, very nearly if not quite as well as the two equations given by the author. To do this, the writer subdivided the points shown by the circles in Fig. 7 into seven groups. The centers of gravity of these groups were then found graphically and their co-ordinates tabulated. The coefficients were then plotted against the squares of ratios, $\frac{H_3}{D_3}$.

It was found that a straight line passes practically through all except the lowest of these points. An equation could be written giving a reversal in the direction of curvature so as to make the curve pass through the center of this lower group and near the lowest isolated point by introducing a term in the form, $C\left(\frac{H_3}{D_3}\right)^n$, but there are not enough experiments with very small differences in depth to warrant this. The resulting equation is:

$$C = 0.877 + 0.32\left(\frac{H_3}{D_3}\right)^2.$$

This, however, is probably not the correct expression for the complete law of the variation in C, which should approach unity instead of a quantity less than unity, for very small values of $\frac{H_3}{D_3}$. For as H_3 decreases, it approaches the value of the velocity head as its limit. With any given value for Q, H_3 decreases with increase in A, which has the area of the channel of approach as its limit. In order that the value of Q may remain constant as H and A both approach their limits, it is necessary that C approach unity as its limit. It seems probable, therefore, that the variation of C is only approximately linear even for sharp edged contractions. In other words, the author's investigations, by demonstrating the variation of C, also demonstrate that neither of the formulas discussed represents the true law for the discharge of contracted sections.

The above simply illustrates another method of analyzing the results obtained and is not intended as a criticism of the author's investi-

Mr. Goodrich.

gation. In fact, he brings out several characteristics in the formulas which might have been overlooked if he had used the method mentioned.

In discussing the sharp edged contraction, the author shows a very interesting relation between the d'Aubuisson and Weisbach formulas. In this connection the writer wishes to call attention to the statement made at the bottom of page 1176, that all the particles below the tail-water level have the same velocity. This assumption is used in the development of the Weisbach formula, but it does not agree with observations of velocities in open channels either above, below, or at gate openings and contracted sections.

The tendency of velocities in a vertical section of a stream to vary as the ordinates to a curve of more or less parabolic or elliptical form is too well known to be discussed here. It might be noted, however, that the rate of change of velocity is especially rapid near the bottom. The writer has measured velocities with a current meter above an open sluice-gate which showed a maximum velocity of approach of approximately twice the average, located considerably below mid-depth. The vertical velocity curve when plotted looked like an ordinary velocity curve inverted. All the observations taken at the several sections made showed the same conditions, and such "under tows" have been frequently observed. It is suggested that the words "theoretical velocity" would have given a more accurate statement for the author's purpose.

The author calls attention to the fact that the coefficients for the discharge above and below the level of the tail-water might be different, instead of being assumed the same as in the ordinary Weisbach formula, and further that the coefficient for the discharge below the level of the tail-water is the larger in the case of contracted sections, while for submerged weirs the larger coefficient applies to the section above the tail-water level. That this difference is due, at least in part, to the effects of contractions on the sides and surface, as explained by the author, is probably true. It may also be true that the depth of the thread of maximum velocity may influence these coefficients. It is certainly true that the maximum velocity and, therefore, the maximum discharge per unit depth, is nearer the surface above a weir than it is above a submerged gate; and the discharge through a contracted section is very similar to that through a partly submerged gate.

The high values of the coefficients for the discharge through both the short and the expanding flume are very noticeable and lead one to think that the coefficients used in practice are frequently too small. The author's discussion of the experiments on these flumes is most interesting, and his theories appear to be most reasonable and logical. They agree with some of the writer's observations and offer explanations for various phenomena for the first time, so far as the writer is aware. A case in point is discussed in Appendix II.

Mr. Goodrich. The writer has not devoted much study to the phenomena of flow through flumes and, therefore, is unable to discuss that part of the paper as he would wish. The paper is one to be highly commended and it is sincerely hoped that the author will soon be able to continue and enlarge upon his investigations, which have added materially to the knowledge of the undeveloped science of hydraulics.

Mr. Lane. E. W. Lane,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been much pleased by the discussions on his paper and, particularly, by the constructive spirit which characterized them. He has always believed that a large part of the value of the papers presented to the Society results from the discussion by the members. No substantial disagreement was expressed with the conclusions, as given on page 1201.

The writer is pleased to note that Mr. Lyle's experience with float gauges agrees with his own. It is probable that they will come into wider use in the future, as their advantages are appreciated. Mr. Lyle evidently had in mind the stream-line theory, as used in certain text-book explanations of hydraulic principles, when he criticized the writer's reference to "the change from stream line to turbulent flow". The stream-line flow referred to is that which takes place at very low velocities, and is an observed fact, not a theory.†

Mr. Henny's experiments with a simple type of Venturi meter are interesting and show the high degree of accuracy which may be attained with such a device. The advantages of the flume type over the enclosed meter are that the former would be free from trouble due to floating trash, and, for most conditions, would require only one reading of head, rather than two.

Mr. Kennison rightly calls attention to the fact that the "undulating surface profile does not represent the pressure head on the water beneath the surface." The "loss of head" in Figs. 11 to 15 and Plates XIX to XXIII, was computed at each observation point in the manner ordinarily used in applying the Bernouilli theorem, that is, using the surface elevation of the water as the "elevation head" and the observed cross-sectional area in computing the "velocity head." The results obtained, in some cases giving negative loss of head, illustrate the danger of this procedure where stationary waves occur. Because of centrifugal force, due to the curvilinear motion of the water particles, pressures are induced in the water which make the pressure head of a point beneath the crest of the wave less, and that beneath the trough greater, than the elevation of the surface would indicate. This has been demonstrated experimentally in the investigations on concrete ships.‡

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* Lafayette, Ind.

† *Phil. Transactions*, 1883; *Proceedings*, Royal Soc., Vol. 174.

‡ *Engineering News-Record*, Vol. 81, No. 23, p. 1019 (December 5th, 1918).

Mr. Lane.

Since the paper was published, the writer has made some studies which tend to show that the coefficients of the flume with rounded entrance and of the expanding flume are due to the curvilinear motion of the water particles in flowing through them. This, also, seems to explain why the flow takes place at less than the two-thirds depth. He hopes to publish these studies at a later date, when more experimental evidence is available.

In answer to the question raised by Mr. Goodrich regarding the curve used in the sides of the expanding flume, it may be said that the dimensions of the flume were chosen rather arbitrarily, the length, 10 ft., having been used as sufficient to give a gradual reduction of velocity, and the width at the lower end, 3 ft., to give small velocities at this point. A curve of such form that it would start tangent to the parallel sides of the flume at their lower end, and give a width of 3 ft. at the end of the flume, was chosen, the parabola having been selected arbitrarily, although an elliptical curve might have been as satisfactory.

The single formula, $C = 0.877 + 0.32\left(\frac{H_3}{D_3}\right)^2$, developed by Mr. Goodrich for the round-edged contractions, is, perhaps, a better working equation than the two given by the writer. The latter were intended primarily to demonstrate the principles of flow involved, which, as has been noted, are not shown by the single formula.

It is possible that the coefficient for contractions approaches unity as the value of $\frac{H_3}{D_3}$ approaches zero. The experimental evidence gives no indication of this, however, and to do so would involve so rapid a change in the coefficient below $\frac{H_3}{D_3} = 0.1$, that it seems very unlikely.

The shape of the vertical velocity curve in the channel of approach would undoubtedly exercise some influence on the flow through the contraction, particularly when the velocity of approach is high. Its influence on the flow of water over weirs is known to be great. In the writer's experiments, however, the velocity of approach was low, and the head equivalent to this velocity was such a small proportion of the total head involved that its effect probably was not material.

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Paper No. 1451

THE "LIGHT RAILWAYS" OF THE BATTLE FRONT IN FRANCE*

By Frank G. Jonah,† M. Am. Soc. C. E.

With Discussion by Messrs. William Wren Hay, F. W. Green, William M. Black, William J. Wilgus, H. F. Dunham, Morris E. Pumphrey, Hazen L. Hoyt, Jr., W. B. Poland, Marshall R. Pugh, Francis W. Perry, Gavin Hadden, and Frank G. Jonah.

Synopsis.

This paper describes the most recently developed method of transportation in the advance sector of an army.

The organization and execution of this work in the American Army was almost entirely in charge of engineers from civil life, many of whom were members of the Society. The object of this paper is to describe their activities and to present to the Profession one feature of the many engineering works that were executed so successfully by the Americans in France.

The following notes are the results of the writer's experience and observation in the construction and operation of "light railways" while acting as Major of the Twelfth Engineers (Railway), U. S. A., assigned to the British Army, and serving on the Picardy front (see Fig. 1) from August 21st to October 27th, 1917, and from that time until December 16th, 1918, as Chief Engineer of the Department of Light Railways of the American Expeditionary Forces.

The "light railway" was a development of the great war, and was gradually extended in scope and importance until, at the close of hos-

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tilities, the mileage of the lines ran into the thousands. These railways were among the principal agencies of transportation, having been, in fact, the link between the standard-gauge lines and the trenches.

It was found by the British, in 1916, that the transportation of supplies, essential for the dense concentration of men at the front, could not be accomplished satisfactorily on the highways, and that some supplementary means of conveyance were necessary; therefore,

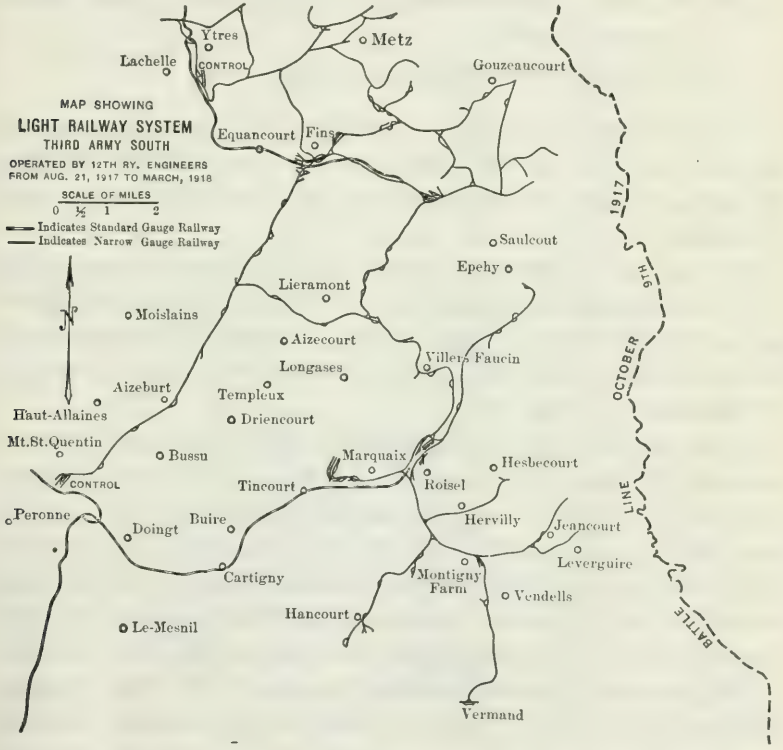

FIG. 1.

orders were given for the development of a comprehensive system of "light railways" for the following primary reasons:

1st.—To relieve the highways of traffic. In addition to the wear and tear on motor transport and the excessive use of gasoline, the wear on the roads was leading to a heavy traffic in road material, so that a large part of the traffic hauled by motor trucks was road material (crushed stone or gravel)

to repair the damage which their own traffic was creating; and, consequently, there would be a great saving in labor in maintaining the roads if other means of transportation were developed.

2d.—To assist in a rapid advance over shell-torn areas when the roads were useless for motor transport and little better for horse transport.

3d.—To convey road material for the rapid repair of roads in the destroyed zones.

4th.—To reduce manual labor at the front.

The lines were developed so completely that, eventually, nearly all heavy material was transported by the "light railways", leaving the highways clear for light, fast-moving automobiles, trucks, and ambulances.

The rail-head, or terminus, on the standard-gauge line was usually 8 or 10 miles back of the front line—far enough to be out of range of the enemy's ordinary shell fire. Between the rail-head and the front, the standard-gauge line was generally destroyed by enemy operations, oftentimes as he retreated.

At the rail-head were concentrated the supplies for a certain sector, consisting of ammunition, rations, forage, road material, building material, fuel, barbed wire, fence pickets, concrete material, culvert pipe, plank, track material, tools—in short, everything necessary for the sustenance of troops and for their means of offensive and defensive operations; and it was the function of the "light railways" to carry these supplies to the forward battery positions and front line trenches.

The lines were also used in moving troops up to the front, bringing back the relieved troops to their rest areas and the wounded from the front, and extensive use also was made of them in salvage operations. The extent to which they were developed can be readily expressed by stating that for every mile of front which an army held, there were ten miles of light railway supplying it.

The gauge of track used by all the armies on the Western front, including the German Army, was 60 cm. (23⅝ in.), usually laid 2 ft. by the British and Americans. There was great advantage in this uniformity of gauge, as it permitted interchange of equipment among the

PLATE XXVI.
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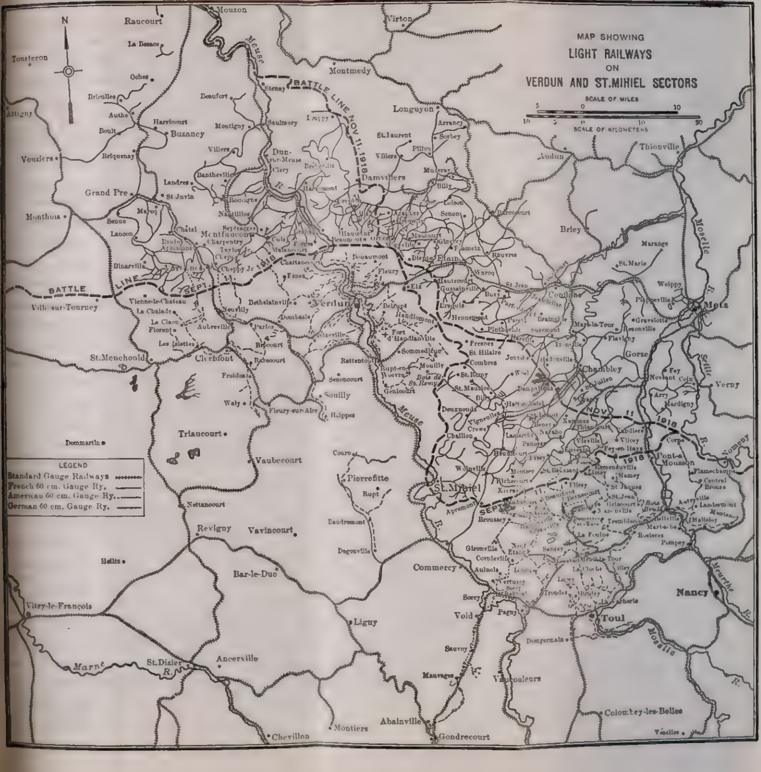

Allies and the immediate use of the captured German tracks after successful offensives.

In a general way, the characteristics of the lines on the different army fronts were similar. Grades of 3% and curves with a radius of 30 m. were freely used to facilitate rapid and easy construction. Embankments were made with 9-ft. crown cuts, with a 14-ft. base at sub-grade.

The French track was of built-up sections, 5 m. long, the rail weighing 9 kilos per meter, with steel ties, 1 m. long, riveted to the rail.

The British used a rail weighing 20 lb. per yd. A certain amount of track was laid in short sections bolted to steel ties, but the greater part was laid with rails at least 24 ft. long, on wooden ties, 4 by 6 in. by 4 ft. 6 in., as it was found that track laid on wooden ties was much more easily kept in line and surface than that laid on steel ties.

The American track was generally constructed on wooden ties, of rail weighing 25 lb. per yd., in 30-ft. lengths. A certain amount of track in built-up sections, with a rail, 5 m. long bolted to steel ties, was kept on hand for use at the extreme front. Sections of track in this length could be handled by a squad of men and coupled up without making any noise, and consequently were preferable for work near the trenches. Such sections also had advantages when building at night.

The German lines were built with rail weighing about 17 lb. per yd. on steel ties, bolted to the rails. They were extremely well built, and by reference to the map (Plate XXVI) showing the "light railways" on the Verdun and St. Mihiel Sectors, it will be seen that they had a greater mileage than the French and American Armies. Their extensive use of these railways was due to the fact that they were short of rubber tires and gasoline, and could not depend on motor transport to the extent that the Allies did; but they were in a position to develop steam railway transportation, as they held the coal and steel districts of France and Belgium.

The Germans had one type of built-up track, which had some advantages. One end of the 5-m. section had a pair of bars with a hooked projection, while the other end had the flange of the rail bent up. In laying track, the hooked bar engaged the bent-up flange of the last section. This enabled the track to be connected on the grade with great rapidity and without noise. Details of this track are indicated in Plate XXVIII.

The speed with which tracks could be constructed by the different Allied armies was practically the same. In September, 1917, the British stated that the construction of 1 mile of track, including ballasting, required 2 100 man days of labor. The "French Manual of Light Railway", issued in April, 1918, states that their average had been ¾ m. per man per day, which is equivalent to 2 146 man days.

Most Important American Line.

On the American front, the most important line, and the best constructed, was the connection between the central shops and storage yards at Abainville and the front line system at the Sorcy rail-head (see Plate XXVII), a line 18½ miles long, on which the average speed of construction was 2 640 man days per mile. As this line, together with the shops and yards at Abainville, and the Sorcy rail-head, was the most important light railway job constructed by the American forces, and as several interesting engineering problems were presented in their construction, they will be described in some detail.

When the American troops first went into the line in the St. Mihiel Sector, the base from which their supplies were drawn was Toul. As the French had troops in the line immediately east of the Americans, and were also using Toul as a base, it became congested, and the French suggested that the Americans build an independent rail-head at Sorcy, 11 miles west of Toul, which was done. This rail-head had the necessary facilities for transferring material from broad gauge to narrow gauge, and from car to motor truck; also warehouses, platforms, etc. It was regarded as very convenient, and represents a typical lay-out (see Plate XXVII).

From this rail-head at Sorcy, the American forces built a 60-cm. line to Cornieville (5 miles), where connection was made with the French system of light railways radiating from Toul. They also took over the operation of the French lines on the west half of the sector, St. Mihiel-Pont-a-Mousson.

The location for the central shops and storage yards of the "light railway" system was a matter for careful deliberation. Many sites were examined and reported on, the objections to most of them having been that they were either too close to the front, too far to the rear, too far to the left, or too far to the right. It was necessary to find a location that would afford ample space for a large yard, with standard-

PLATE XXVII.
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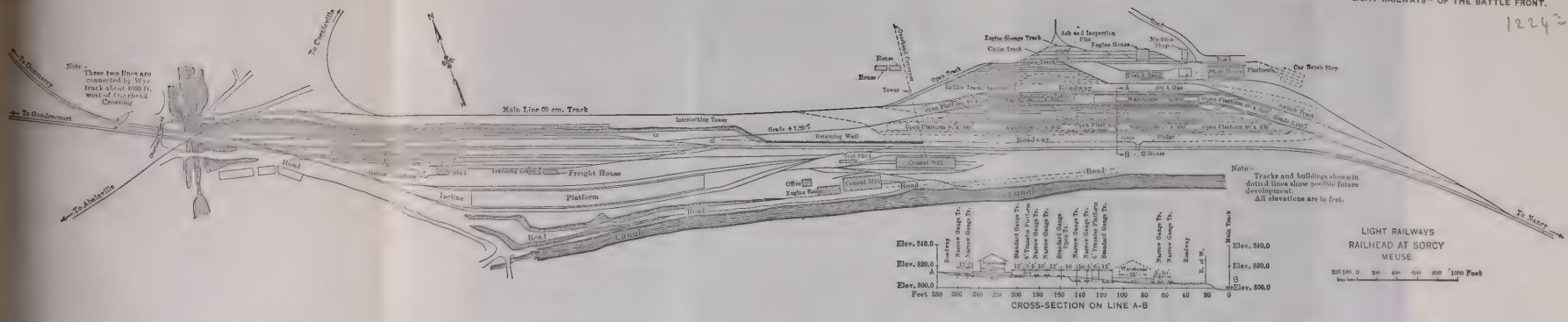

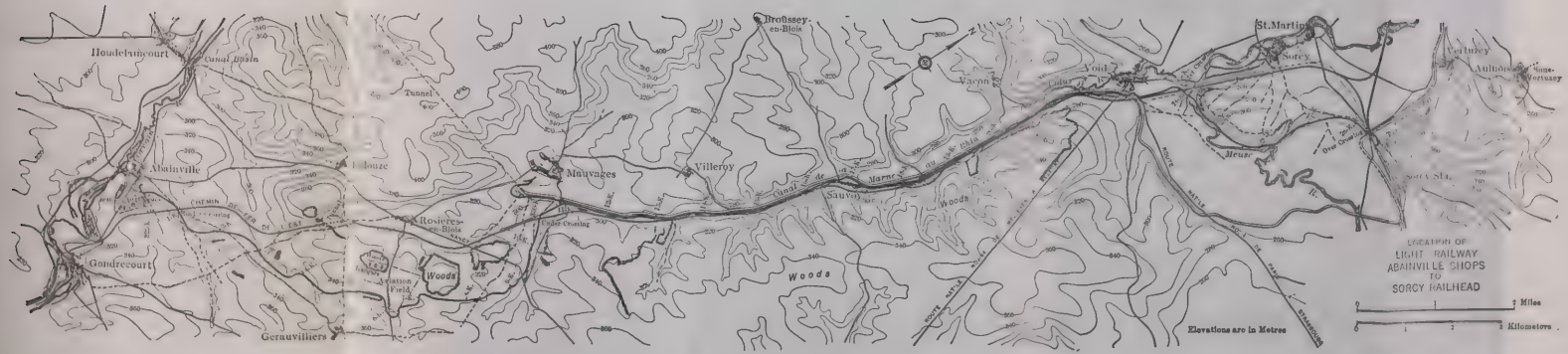

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PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
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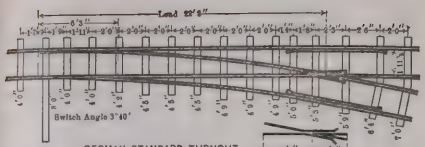
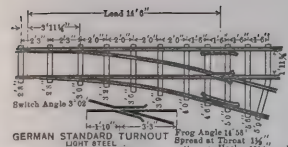
JONAH ON
"LIGHT RAILWAYS" OF THE BATTLE FRONT.

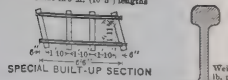


GERMAN STANDARD TURNOUT



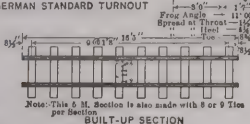
GERMAN STANDARD TURNOUT
LIGHT STEEL

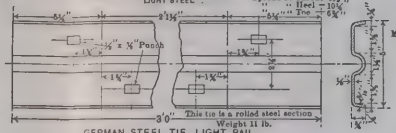
Note:-This type of section is made only in 3 M. (6'6") lengths
All other built-up sections are made in 5 M. (16'5") lengths


SPECIAL BUILT-UP SECTION

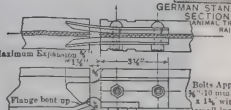
Weight 11 lbs. per yd.
GERMAN STANDARD SECTION
ANIMAL TRACTION RAIL

Weight 17 lbs. per yd.
GERMAN STANDARD SECTION

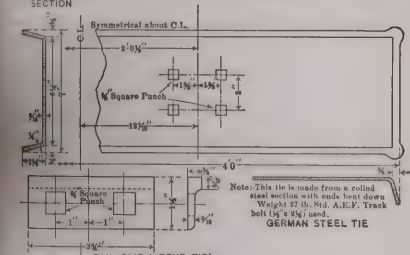

Note:-This 6 M. Section is also made with 8 or 9 Ties per Section
BUILT-UP SECTION

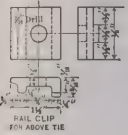


GERMAN STEEL TIE, LIGHT RAIL

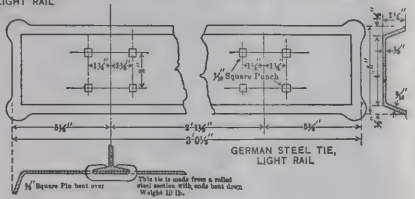


DETAIL OF CONNECTION USED ON ABOVE BUILT-UP SECTION

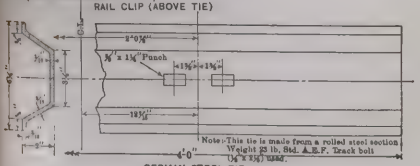

Symmetrical about C.L.
⅞" Square Punch
Note:-This tie is made from a rolled steel section with ends bent down Weight 27 lb. Std. A.E.F. Track bolt (1½" x 3¼") used.
GERMAN STEEL TIE



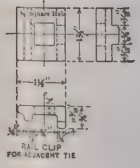
RAIL CLIP FOR ABOVE TIE



GERMAN STEEL TIE, LIGHT RAIL
DETAIL OF RAIL FASTENINGS



GERMAN STEEL TIE



RAIL CLIP FOR ADJACENT TIE

GERMAN LIGHT RAILWAY
DETAILS OF TRACK
FROM DATA OBTAINED ON CAPTURED
GERMAN LINES ST. MIHIEL SECTOR

gauge rail connection, and preferably canal connection as well. It was desirable also to have a location, at least, 30 km. back of the front line, as prisoners of war could be used if they were kept back that far; however, the Americans did all their work with their own troops.

Abainville, 2 km. north of Gondrecourt, was recommended by the writer, in March, 1918, as a suitable location for shops and yards, which recommendation was approved. Subsequent events proved it to be a most desirable location, as it was directly behind the St. Mihiel and Argonne fronts, on which the greater part of the American operations were conducted. Abainville was 35 km. back of the line, and was not within the area usually bombed by the Germans.

The shops were designed for the repair of "light railway" locomotives and gasoline tractors and for the assembly and repair of freight equipment; and the yards were designed for the storage of track and bridge material. The whole plant covered an area of 116 acres. The track arrangement was such that material coming up from the ports on standard gauge was unloaded in the storage yard and subsequently taken to the front, as required, on the "light railways".

The shop buildings consisted of the Austin type of steel building, and were erected by the American troops under the direction of a superintendent of the Austin Company, who had been sent over for that purpose. The buildings were of steel framework, boarded in with lumber obtained in France. The roof was boards covered with prepared roofing. The buildings were generally 40 ft. wide, 200 ft. long, and included an erecting shop, machine shop, gas-engine and auto repair shop, planing mill, two storehouses, car shop, foundry, blacksmith shop, power-house, and oil house.

Skillful Location Required.

In the construction of the line leading from these shops to the Sorcy rail-head, it was stipulated by the French authorities that there should be no grade crossings of the standard-gauge railways and no interference with navigation on the canals. These requirements necessitated skillful location.

The line was projected on the French standard contour maps 1 to 20 000, a study of which showed the feasibility of a line with grades not to exceed 3 per cent. An examination of the route showed that the necessary crossings of standard-gauge tracks could be made under

the main line viaducts or through large culvert openings. These points were selected as the governing conditions of the location, and the line was run accordingly. It was found that the French maps were extremely accurate—the summit elevations checked out exactly—the grade line and work was just what was expected.

It was necessary to cross the Marne-Rhine Canal at Void. The limited room for the 60-cm. line in this town necessitated crossing the canal at an angle of 68°, requiring a span of about 28 ft. to preserve the standard 6-m. width of waterway at right angles. Fortunately, the American engineers had some bridge stringers 8 by 16 by 28 ft., which had been sent over from the United States. The writer designed a counterweighted lift bridge (see Figs. 2, 3, and 4) with this material, which could be operated by one man. This proved so successful, that a standard plan was made for lift bridges, and as it was expected that most of the canal crossings could be constructed at right angles, a complete bridge was built and kept in reserve, the details for which are shown in Plate XXIX.

At Sorcy, it was necessary to cross the main line of the Chemin de Fer de l'Est, and as it was near the junction of the line from Gondrecourt with the main line, four main line tracks were crossed (see Figs. 5, 6, and 7). The railway at this place crossed over the canal, and the 60-cm. line was located to cross the standard-gauge tracks and canal at the same place. Fortunately, this crossing occurred in a cut, so that the line could be carried across with the minimum of bridging.

No girders or spans were available for this crossing, and it was necessary to design and build a bridge with the available material. It was found that two deck Howe trusses, with a few bents of trestle approach, would fit the conditions, give the necessary clearances, and would be the most practicable type of structure. These trusses were built of 5 by 10-in. road plank, and each truss weighed about 4½ tons. There was a locomotive crane working in the yard, which could lift 5 tons, so that after the bents were erected, the trusses were picked up by the crane and set in place in a few minutes. The whole bridge was framed and erected in five days, with no interference or delay to the traffic on the standard-gauge lines.

The Meuse River was also crossed by this line. At the place of crossing the stream was 125 ft. wide, with 14 ft. of water, and had very little current. Pile trestles were used in this case, as shown by Fig. 8.

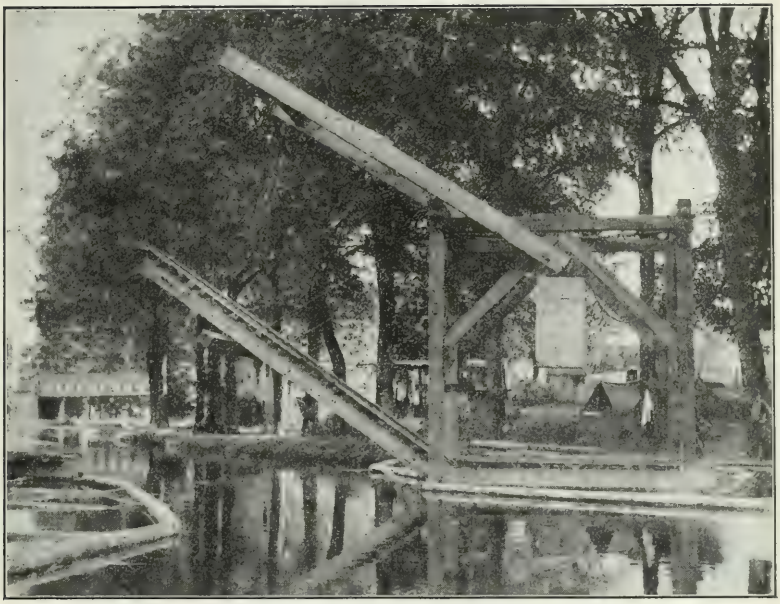


FIG. 2.—LIFT BRIDGE OVER CANAL AT VOID (MEUSE).

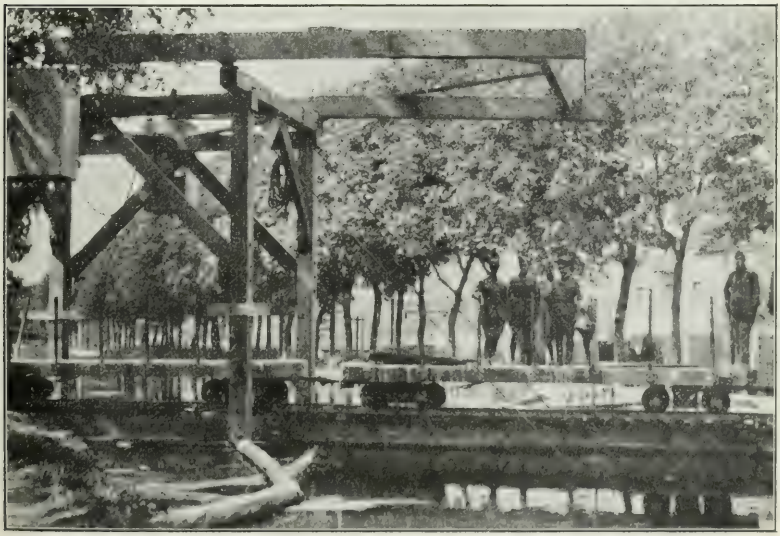


FIG. 3.—LIFT BRIDGE OVER CANAL AT VOID (MEUSE).

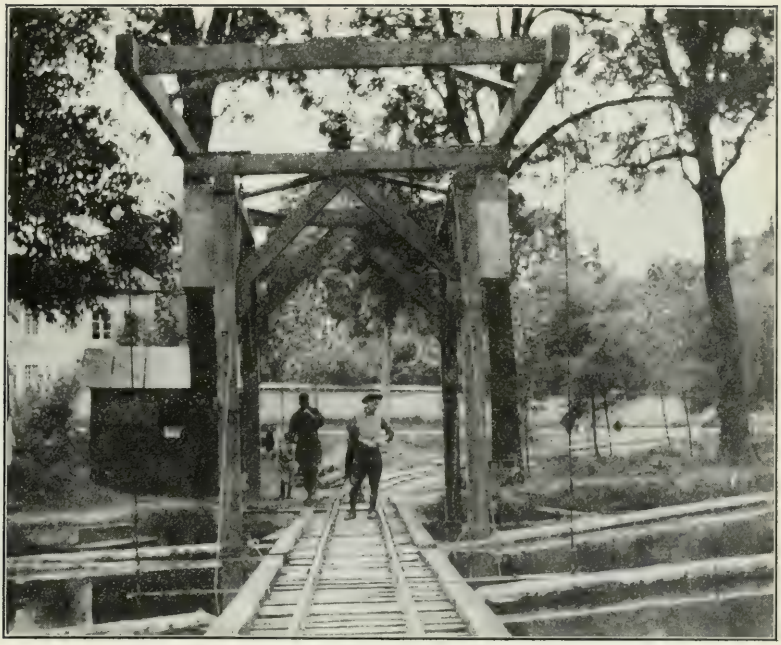


Fig. 4.—Lift Bridge Over Canal at Void (Meuse).

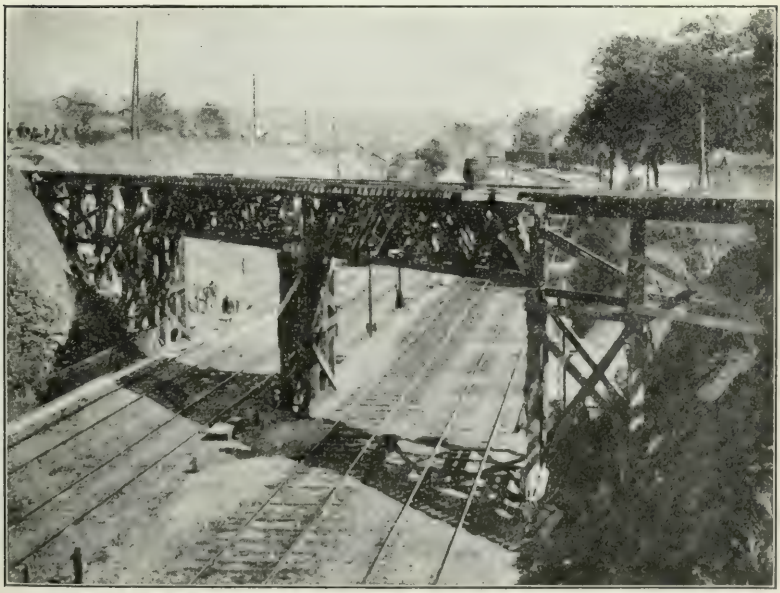


Fig. 5.—Bridge Over Chemin de Fer de l'Est, at Sorcy (Meuse).

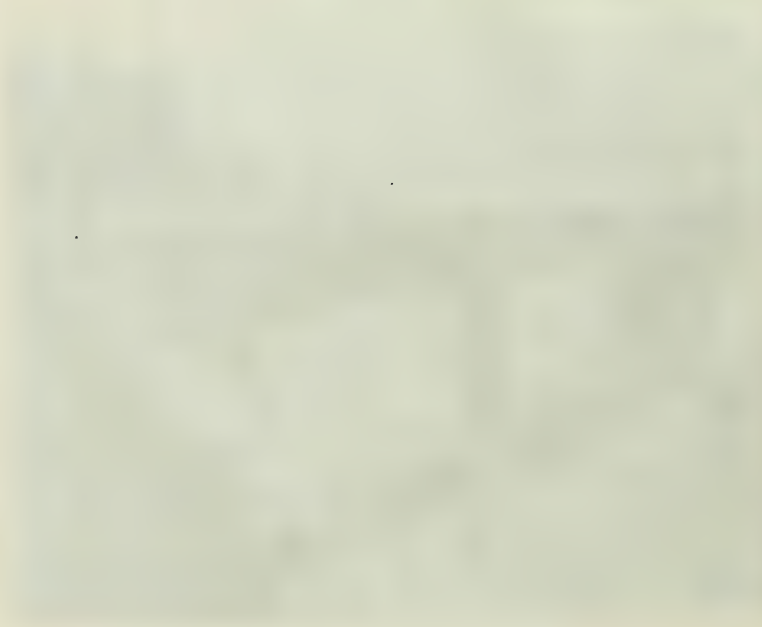


FIG. 6.—BRIDGE OVER CHEMIN DE FER DE L'EST, AT SORCY (MEUSE).

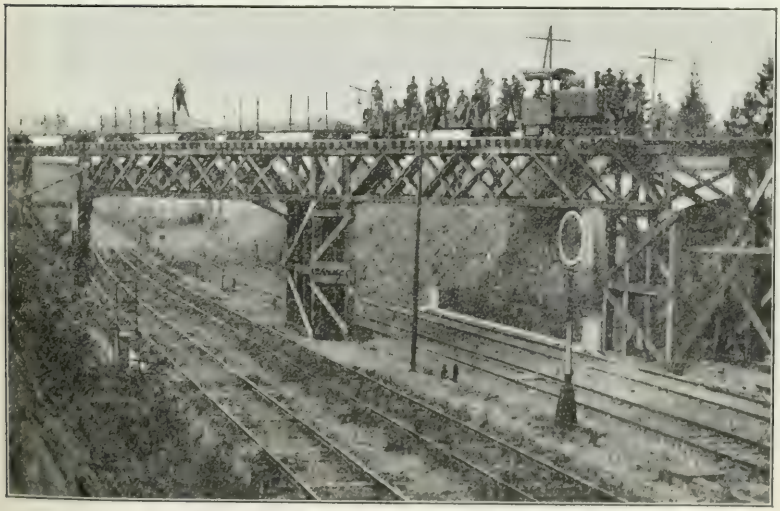


FIG. 7.—BRIDGE OVER CHEMIN DE FER DE L'EST, AT SORCY (MEUSE).

FIG. 8.—BRIDGE OVER MEUSE RIVER NEAR VOID.

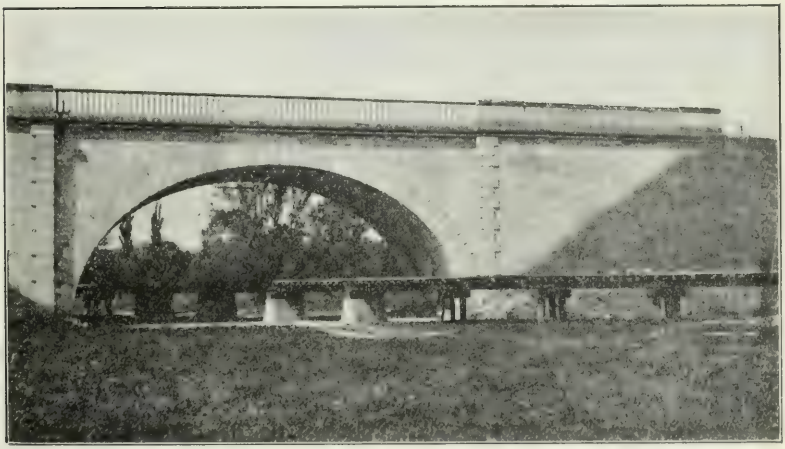
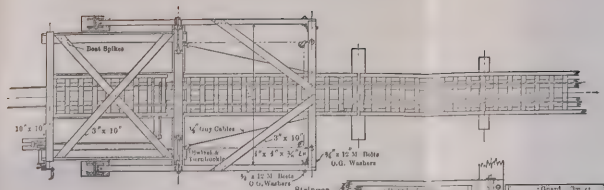
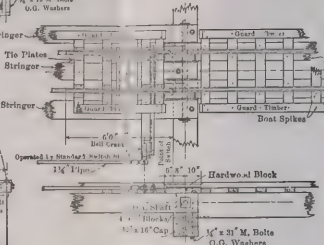


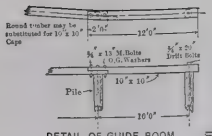
FIG. 9.—CROSSING UNDER CHEMIN DE FER DE L'EST NEAR VOID.



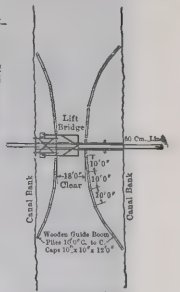
SECTION A-A



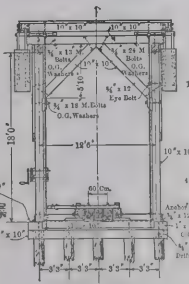
DETAIL OF PIVOT AND SWITCH MECHANISM



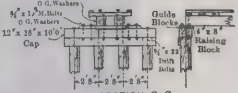
DETAIL OF GUIDE BOOM



SITUATION PLAN



SECTION B-B



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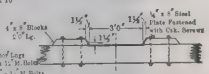
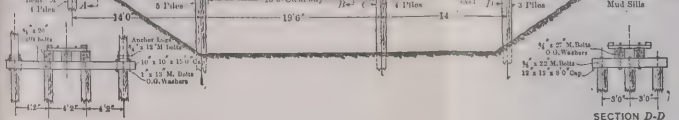


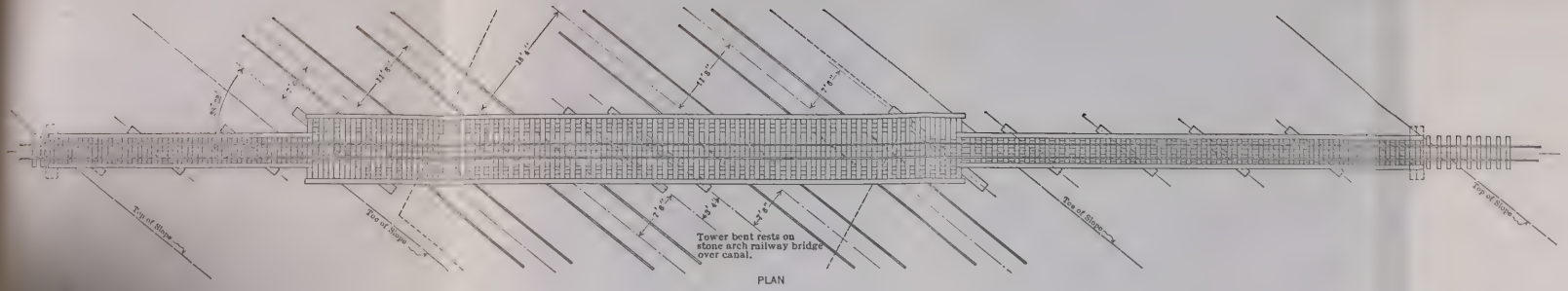
DETAIL OF GUIDE BLOCKS



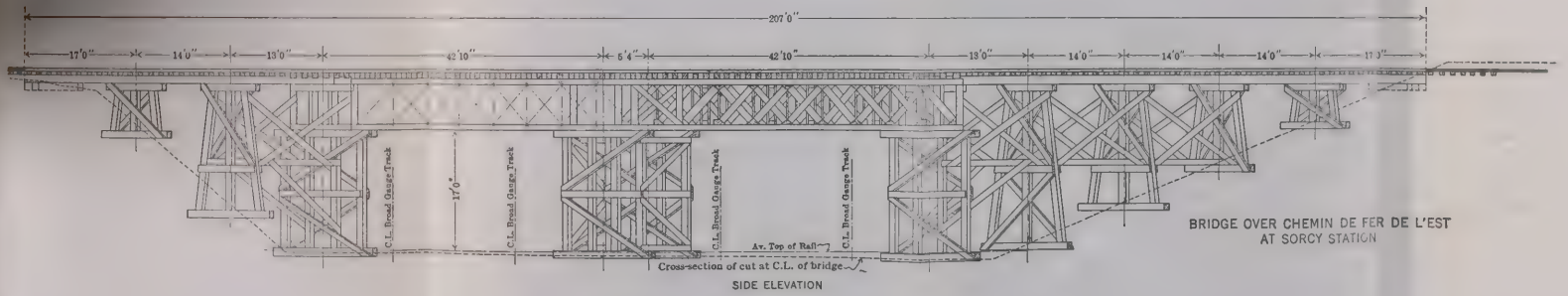
SECTION A-A

SECTION D-D

STANDARD LIFT BRIDGE
FOR
CANAL CROSSINGS AT RIGHT ANGLES



PLAN

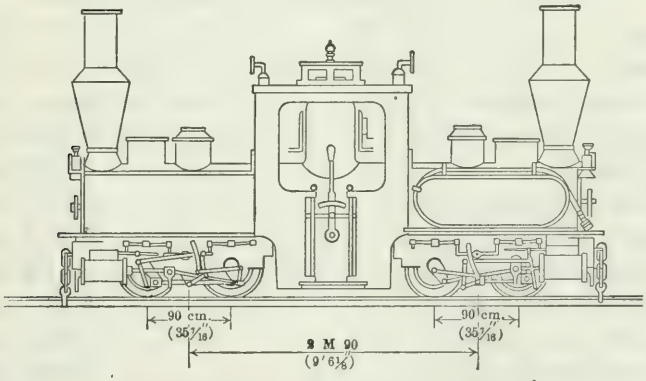


SIDE ELEVATION

BRIDGE OVER CHEMIN DE FER DE L'EST
AT SORCY STATION

The main line of the Chemin de Fer de l'Est was crossed in the Meuse River bottom through an overflow relief opening. A view of this structure is shown in Fig. 9, which is a good example of the splendid, beautiful, masonry structures which one sees everywhere on the French railways.

The types of engines and cars used on these railways are shown herewith. The Baldwin engine would handle 60 tons of freight up a 3% grade. The steam locomotives were not used up to the point where their smoke and steam would betray their presence to the enemy; some little distance back from the front lines they would be cut off, and the cars were handled to the extreme front by gas engine. The operating rules were extremely simple, and despatching was done by telephone.

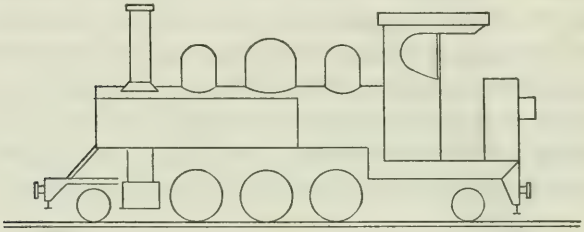


FRENCH DOUBLE END STEAM LOCOMOTIVE
FOR LIGHT RAILWAY
MODEL 1888

TABLE 1—A. E. F. Light Railway Freight Cars.

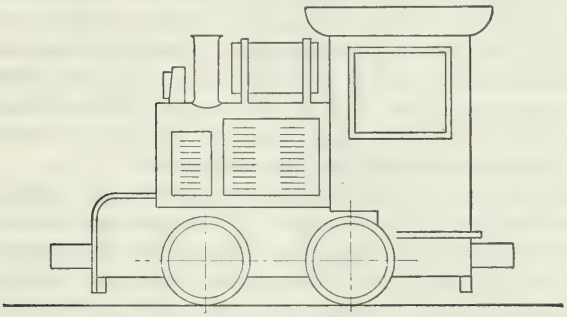
	Box, covered.	Box, open top.	Gondola.	Flat.	Dump V shape.	Tank.
Capacity	600 cu. ft. 22 000 lb.	.....	210 cu. ft. 22 000 lb.	22 000 lb.	27 cu. ft.	2 000 gal.
Weight	10 900 lb.	.....	9 000 lb.	8 000 lb.	1 050 lb.	12 200 lb.
Length, inside	19 ft. 10 in.	.....	19 ft. 8¾ in.	22 ft. 1 in.	5 ft. 8 in.	19 ft. 5½ in.
Width, inside	5 ft. 5 in.	.....	5 ft. 3 in.	5 ft. 3 in.	4 ft. 8 in.	5 ft. ½ in.
Height, inside	5 ft. 8 in.	.....	2 ft.	5 ft. 7 in.	.....	.....
Length, over coupler	24 ft. 1¼ in.	.....	24 ft. 1¼ in.	24 ft. 1¼ in.	6 ft. 9 in.	24 ft. 1¼ in.
Width, over all	6 ft. 6¼ in.	.....	5 ft. 7 in.	5 ft. 7 in.	4 ft. 8⅝ in.	5 ft. 7 in.
Height, over all from rail	8 ft. 4¾ in.	.....	4 ft. 4⅝ in.	.....	4 ft. 5 in.	.....
Height, to floor	2 ft. 4¼ in.	.....	2 ft. 4¼ in.	2 ft. 4¼ in.	.....	2 ft. 4¼ in.
Height of coupler	17 in.	.....	17 in.	17 in.	17 in.	17 in.
Wheel base, truck	3 ft.	.....	3 ft.	3 ft.	24 in.	3 ft.
Distance, C. to G. trucks	15 ft.	.....	15 ft.	15 ft.	.....	15 ft.
Diameter of wheels	15¾ in.	.....	15¾ in.	15¾ in.	14 in.	15¾ in.

A. E. F. Light Railway Steam Locomotive.



Type	2-6-2
Gauge	23⅝ in. (60 cm.)
Working pressure	178 lb. sq. in.
Curve, minimum radius	65 ft. 7⅜ in. (20 m.)
Driving wheel base	5 ft. 10 in.
Total wheel base	15 ft. 7 in.
Length between couplers	21 ft. 7 in.
Height of coupler, rail to center line	16½ in.
Width, maximum	6 ft. 5 in.
Height, maximum above rail	9 ft. 3 in.
Weight on drive wheels	23 500 lb.
Weight on front truck	5 000 lb.
Weight on rear truck	5 500 lb.
Total	34 500 lb.
Tractive effort	6 225 lb.
Ratio of adhesion	3.7
Coal capacity	1 700 lb.
Water capacity	486 gal.

A. E. F. Light Railway Gas Locomotive.



Type	4-wheeled, 4-cylinder, 4-cycle, vertical, water-cooled
Gauge	23⅝ in. (60 cm.)
Wheel base	4 ft. 0 in.
Horse-power	50
Fuel capacity	30 gal.
Length over all	13 ft. 0 in.
Width over all	5 ft. 2 in.
Height over all	8 ft. 5 in.
Height of coupler, rail to center line	16 in.
Weight, total	14 000 lb.
Transmission	2 sets of speed change gears
Draw-bar pull (Level track)	Low gear, 3 000 lb. High gear, 1 500 lb.
Speed	Low gear, 4 m. p. h.; high gear, 8 m. p. h.
Brakes	Hand brakes to all drivers

DISCUSSION

Mr. Hay. WILLIAM WREN HAY,* ASSOC. M. AM. SOC. C. E. (by letter).—It is not generally realized that the so-called "light railway" was carefully studied, and the standards of track and rolling equipment developed in Europe during the late '80s. Originally used by Decauville in his plant at Petit-Bourg, it was proposed by Col. Pechot (then a Captain of Artillery), as a means for the movement and supply of heavy siege artillery, and was the object of extended experiments and studies conducted by Col. Pechot, with the collaboration of M. Bourdon, a Government engineer, during the period 1881-89. The name "Pechot" survives in the Pechot locomotive, Type 1888, in the system of multiple-axle trucks used under heavy artillery pieces, and in various manufacturers' catalogues in the track described as "System Pechot."

In 1882-85, the track now known throughout the world as "Decauville" was developed by the company of that name and has served as the model for most of the track made in Europe, and its influence is readily traced in the United States. The system developed in France served also as a model for the Germans, and after tests conducted by them in 1891, was adopted in 1892, and standardized as Army equipment in 1897.† There is a record that as early as 1879 the Austrians had some narrow-gauge railway equipment for military purposes, but the writer is unable to determine when it was standardized for such use.

The Germans adopted the light railway for strategic use in times of peace, supplementing their standard-gauge railways toward their frontiers, but after trench warfare became fixed in the recent conflict, they developed a very dense system of narrow-gauge supply lines, particularly in front of Verdun and in the St. Mihiel salient, as is well shown on the map (Plate XXVI). So general was their use in the St. Mihiel area, that batteries and billeting areas on the slopes of steep hills were supplied direct from the main lines in the valleys by means of funicular tracks.

Originally, the writer believes, the French Army developed this material for use in connection with siege batteries and permanent fortifications, but it was soon realized that it was also of strategic, or tactical, value, and considerable experience was gained by them in connection with the military railways built in the French Zone, in Morocco. From press reports, the French had at the beginning of the war about 870 miles of 60-cm. track, with 60 locomotives and 350 cars. At the signing of the Armistice, this had been increased to almost 2 500 miles of track, 900 locomotives, and 4 500 cars. The American forces were reported by the Chief of Engineers to have been operating 1 390 miles of

* Forest Hills, N. Y.

† *Le Genie Civil*, Vol. 72, p. 449 (June 22d, 1918).

track, of which about 1 080 were captured German lines, 165 locomotives of all classes, and 1 695 cars. The writer has seen no statement of the mileage of either the British or German lines. Mr. Hay.

Contrary to the text, both the French and German standards for portable track are for rail weighing 9.5 kg. per m. (about 19 lb. per yd.), in both cases attached to eight rolled steel ties per 5-m. length, the German track being clip-bolted as shown, the French cold-riveted to the ties. The usual French rail has a greater base than height, while the German rail has a greater height than base. During the war, many sections were in use. These can be compared with the equal base by height standard 20-lb. rail used by the English, and the standard American 25-lb. section. The axle concentration on this track was not allowed to exceed 3 500 kg. for 0-6-0 type locomotive loading. (In this connection, the conclusion reached by British engineers in India was to allow 3 long tons—about 3 000 kg.—for the concentration on 20-lb. rails, 24-in. gauge track).

The Pechot locomotive has received a great deal of publicity since the opening of hostilities in 1914. It was originally developed for use in and around fortifications, where sharp curves and steep grades were necessary, and probably rendered its most notable service on the steep hillsides in and around Verdun, the very service for which it was intended. The trucks are of the suspended, articulated type. The firebox is double, with a single grate, and designed so that for all grades less than 10% it is constantly submerged. The interesting thing to the general public is that this locomotive is double and "works both ways". The grate area is too small for heavy service. These locomotives are very expensive compared to other types.

A locomotive of the Mallet articulated type, 0-6-6-0, was built in Europe before the war and received some use in Morocco and on fortification construction, but whether any was used on the light railways of the battle front the writer never heard. The majority of the locomotives used by the French were the Decauville 8-ton (empty), 0-6-0 type, developed from the experience gained in Morocco.

F. W. Green,* M. Am. Soc. C. E. (by letter).—The author is to be congratulated on the thorough and comprehensive manner in which he has treated this interesting subject. Although much has been written on various other activities incident to the conduct of the war, very little has been done in the way of description of the construction and operation of the "light railways." This paper, therefore, presents to the Profession a subject comparatively new; and although it relates to the application of the light-railway principle to military operations, the civil engineer will not fail to observe its availability for a number of purposes, in which it may prove to be more economical than practices heretofore followed. Mr. Green.

* St. Louis, Mo.

Mr. Green. While serving with the Third British Army as an officer of the Twelfth Engineers, in an endeavor to discover the *raison d'être* of the "light railways", the writer was informed that the experiences of the winters of 1914-15 and 1916-17, especially the latter, clearly indicated the danger of military reverses involved in an adherence to the use of motor lorries for transportation between rail-heads and battle front. With the unprecedented density of men in the battle lines, transportation by motor transport involved chronic congestion, with consequent decrease in mobility, as well as the diversion from combat duty of large numbers of men required to keep the highways passable. It was felt that the saturation point had about been reached when the damage done to roads by motor trucks hauling road repair material almost offset the benefits accruing to the road by the use of said material.

In casting about for means of ameliorating these conditions, attention was called to what the French styled the "Decauville" railway, usually of 1-m. gauge. The British decided to adopt a gauge of 60 cm., because of considerations of greater mobility, less cost, and inability of the enemy to connect up with it and use it in case of retirement before it could be removed.

The writer recalls having observed the use of light railways in certain large manufacturing plants, notably steel mills, as long ago as 25 years. It is not known whether that was another application of the "Decauville" principle, or a mere coincidence.

Early in 1917, light railways were started experimentally behind one of the British Armies. Their success was so immediate that by the summer of that year the larger percentage of transport between broad-gauge rail-heads and the battle front was changed from motor lorries to the light railways. The writer was informed by a British officer that about 18 miles of single-track light railway operation in the Gouzeaucourt sector permitted the abandonment of the operation of more than 600 motor lorries and the retirement from road maintenance labor forces of more than 3 000 men.

In the back areas, steam locomotives were used; where under enemy observation day and night, gas or gas-electric tractors were utilized. In the design of the steam locomotives, the saddle type of water tank was provided, presumably because of the additional adhesion available for tractive effort. Whatever advantage thus accrued was, in the writer's opinion, more than offset by the decreased stability against overturning resulting therefrom. As these locomotives were "shopped", the saddle tanks were sometimes removed and mounted on a trailer type of tender behind the locomotive, and it was remarked that the tendency to derailment was very much reduced.

Mr. Green.

It is believed that there is a large field for the development of this type of transportation in the United States. A great amount of highway construction is now under way in many States. The transport of road material from broad-gauge railway cars to the point required on the work by gas or gas-electric tractors and self-dumping equipment should prove much more economical than by the use of teams. Similarly, there are large areas of hardwood timber suitable for railway cross-ties and other purposes, which heretofore have been considered beyond the limit of commercial exploitation. The light railway, cheap of construction and maintenance, the track of which can be used over and over again with small cost of removal, ought to find a field for development in such cases.

The author undoubtedly gave much more thorough consideration to developing his design for the rail-head at Sorcy than is usual under war-time conditions. As a result he succeeded in obtaining the most complete and practicable layout the writer has ever seen. In the stress of war it is quite common for such layouts to be made to fit the ground with a minimum of construction effort, without regard to convenience and facility of operation.

In the design of the lift bridge at Void, the author has demonstrated not only the resourcefulness with which American civil engineers applied themselves to the solution of military problems, but also that Mars and Apollo follow different paths.

The Howe trusses over the Eastern Railway at Sorcy, however, are evidently the result of very careful and conscientious designing, and when considered from the standpoint of military engineering, are unusually pleasing to the eye. The location of the line from Abainville to Sorcy also shows that much thought and study was given to it, and Col. Jonah is entitled to much credit for the successful results obtained.

Mr. Black.

William M. Black,* M. Am. Soc. C. E. (by letter).—In the introduction to his paper, the author states that the execution of the light railway work was almost entirely in charge of engineers from civil life. At this time when the necessity for universal military training for our citizens is under consideration, and, further, while some members of the Society are advocating the removal from the charge of the Corps of Engineers of the Army of the civil works from which alone the officers of that Corps were enabled to obtain practical training in time of peace in the organization and conduct of important engineering works—with the development of character which results from such responsibilities—certain comments on this statement are pertinent.

At the outbreak of the war, in accordance with the traditional policy of our country, the United States had available a pitifully small number of trained soldiers by whom her great man power could be instructed and made useful for the struggle into which she had been forced.

* Maj.-Gen., U. S. A. (*Retired*), Washington, D. C.

Mr. Black. On April 10th, 1917, there were only 256 engineer officers in the Regular Army. In the National Guard, the Engineer Service had been neglected, and even in the few engineer units of that service many of the officers were not engineers. During the war the number of regular officers was increased to 449, partly from graduates of the United States Military Academy and partly from young graduates of civil technical schools. Of these regular officers, 94% were engaged in military duty during the war. Of the remainder, the majority were physically unfit for field work. They, assisted by engineer officers on the retired list and the United States Assistant Engineers, kept the civil works in charge of the Corps in uninterrupted operation. At the outbreak of the war, the total enlisted strength of the Engineers of the Regular Army was 2 228, organized into 15 units. The necessities of war required the expansion of the above to 473 units, comprising 284 983 men. The number of commissioned officers was increased to 10 886.

It is manifest that this great expansion could not have been accomplished efficiently without the aid of the civil engineers of the country. This aid was rendered promptly and willingly, and the final success of our operations in France is manifestly due greatly to their efforts; but is there a man of all this number who did not regret that he had not had previous training, at least in matters of army organization and administration? Was it not a difficult task for many to merge their individualities in the whole and to learn to act simply as one cog of the great military machine? Is there one who did not recognize the waste of life and treasure which follows the entering of a nation into war without proper preparation and prior training of her man power?

The author has given an interesting account of one item of the light railway work of the American forces in France. Before the United States entered into the war the necessity for light railways had been established as an essential part of the Army transportation system. Its zone of operations is from the rail-head beyond which the standard-gauge railways cannot be maintained, up to the firing line, a distance of about 10 miles ordinarily, although in the Toul Sector there was a regular service over a distance of from 26 to 33 miles. When hostilities ended 2 240 km. of light railways were under American Army control, of which about 300 had been originally constructed by the French, but rehabilitated by the Americans, 200 had been constructed outright by the American forces, and 1 740 had been captured from the Germans.

Up to February 1st, 1919, American-operated light railways hauled 860 600 tons of ammunition, forage, rations, water and construction materials and personnel, representing a ton mileage of 8 106 700, including the empty haulage.*

Though the material supplied, both track and rolling stock, proved satisfactory, there is need for further experiment and improvement. In

* Historical Report, Chief Engineer, A. E. F.

some respects the German locomotives were judged superior to those used by the American forces. Mr. Black.

As a measure of economy in administration as well as to provide a means for trying out new devices in material and for giving training to a limited number of officers and enlisted men, the writer, when Chief of Engineers, advocated the installation of a system of light railways at each of the large permanent army camps. Such a system is installed at the Engineer Post at Camp A. A. Humphreys, Va., and its operation has resulted in a material saving of expense as compared with the usual transportation of men and supplies by motor trucks.

William J. Wilgus,* M. Am. Soc. C. E. (by letter).—The writer feels that it will not be amiss to touch on the early development of the Light Railway Section of the American Army's transportation activities in France. In June, 1917, as a member of a commission sent by the War Department to report on the railway needs of the armies of the Allies, the writer had an opportunity of cursorily observing the methods adopted by the British and French on the Western Front; and shortly after, when he was assigned to duty with Gen. Pershing's staff and undertook the task of planning for the transportation of American troops and supplies, he was able to use the information so gained as a basis for estimating the probable requirements in that respect, including "light railways". Mr. Wilgus.

In the absence of a decision as to the intended size of the American forces in France and where they were to fight, the writer arbitrarily assumed that within a year they would total 500 000 fighting men, equivalent to, say, 750 000 men in all branches of the service. Likewise, he assumed that the combat troops would function in the foothills of the Vosges, along a front of some 40 miles centering on Nancy, and that the volume of supplies of all kinds to be transported would average 60 lb. per man per day for all American forces, or 30 lb. per man per day in the advance area for those actively engaged at the front. It should be added that a 40-mile sector was assumed because it was understood that an average of 12 500 troops per mile of front was the maximum that could be effectively employed. Later, it was found that this average was too small and that the point of saturation was somewhere between 15 000 and 17 500 per mile.

A depth of from 5 to 12 miles along the front was taken as the zone within which supplies would be distributed largely by means of light railways, as experience had shown that it was unwise, from the standpoint of safety, to maintain standard-gauge rail-heads nearer than the lesser of these distances, while operating conditions forbade a distance in excess of the greater. The later experiences of the British led them to the conclusion that the light railways could function with

* New York City.

Mr. Wilgus. the rail-head at a minimum distance of 8 miles from the front with nearly the same operating efficiency as at 5 miles, while the safety would be quadrupled. In fact, they considered, as a result of several years of trench warfare, that the depth of the zone to be served by light railways should vary from 8 to 16 miles, instead of 5 to 12 miles, the accepted practice at the time of our entry into the war.

Within this zone it was customary, in general, to have three single-track lines, parallel with the front, and connected with each other and with the standard-gauge rail-heads in such manner that the traffic of the group serving each rail-head would circulate in one direction over a multiplicity of routes, with ample insurance against serious interruption in case any part of the group should be injured by shell fire. The line nearest the enemy was intended for supplying the front-line trenches and light artillery, the second for the medium and the third or rear line for the heavy guns. The standard-gauge rail-heads, at which supplies and troops were transferred to the light railways, averaged a distance of 3 miles apart in the French sectors, each rail-head thus serving approximately 22.5 route miles or 30 single-track miles of light railways, which, in turn, distributed supplies for from 40 000 to 50 000 men.

On the basis of an average of 10 miles of single track, or 7.5 miles of route, per mile of front, it was estimated that materials should be ordered for 400 miles of single track, with steel ties (2 640 per mile), to which 20% was added for various services in the rear and for stock on hand, a total of 480 miles.

The requirements for turn-outs were found to average 10 per mile of single track, and therefore 4 800 sets were ordered, of which one-half each were to be right and left handed. It was designated that the leads should have a radius of 30 m. and that the ground switch-stands should throw parallel to the track.

As stated by the author, rails weighing approximately 20 lb. per yd. were in general use by the Allies, but there was a general feeling, especially among the British, that this was too light for the proper maintenance of track, ballasted as a rule with poor material and laid on soft ground often torn by shell fire. Under these circumstances a stiffer rail weighing 25 lb. per yd. was adopted, even though the heavier weight would be more disadvantageous in laying track. Lengths of rail were fixed at 5 m. with a small proportion of $2\frac{1}{2}$ and $1\frac{1}{4}$-m. lengths for closures at curves; 20% of the rail was ordered curved, of which 70% was to have a radius of 30 m., 15% of 50 m., $7\frac{1}{2}$% of 20 m., and $7\frac{1}{2}$% of 100 m.

The question of ties was troublesome. The operations of the British lay in the flatter region of the North, where the soil was comparatively soft and where the topography was such as to require cuts

and fills; whereas with the French in the higher altitudes rock was at the surface and the terrain was such that tracks could be built on the contour lines with practically no grading. In consequence, the British in many places found necessary the use of such wood as they could pick up for interlacing between the steel ties, especially in shell-torn areas where it was necessary to raft the track over soft fillings. It was felt that the troubles of the British would be obviated by adopting short wooden ties, 4 by 6 in. by 4 ft., of which 115 000 were ordered specially for that purpose. Mr. Wilgus.

The type of fastenings called for careful consideration. The British did not look with favor on the French practice of riveting the ties to the rails because: (*a*) much trouble resulted from irregularity of the fixed gauge due to poor workmanship; (*b*) the rivets were said to rust and break under traffic, with resultant derailments; and (*c*) pre-assembling of the track parts prevented close packing for shipment and, in consequence, was wasteful of vessel and car space in transportation by sea and land. The latter reason, bearing as it did on the transportation factor that was to have so much to do with winning the war, was considered sufficiently forceful to dictate the use of clip fastenings which permitted the close packing of the rails and ties separately, and later their assembling when and where needed.

In the ordering of rolling stock the experience of the British was found to be borne out quite closely by an analysis of the requirements under the above-mentioned conditions.

Counting three gasoline locomotives as equal to one steam locomotive, the calculated needs, based on an average length of haul of 5½ miles, an average daily performance per equivalent working locomotive of 300 ton-miles and a surplus of 40% over actual working needs, were about 0.54 equivalent locomotives per mile of single track or a total of 258. Of these, approximately three-quarters, or 195, were to be steam locomotives. The remainder, multiplied by three, gave the required number of gasoline engines, 189, of which two-thirds were to have a capacity of 40 h.p. and one-third of 20 h.p. It was requested of the War Department that the steam locomotives weigh 3½ tons per axle and that the saddle tanks be hung so as to give a low center of gravity. The failure to observe this precaution with the British was the cause of frequent and embarrassing derailments on soft track.

As to cars, the British authorities gave their average requirements as ranging from 12 to 14 per route mile or 9 to 10½ per track-mile. Calculations based on the aforesaid data and on a daily performance of 27 ton-miles per working unit, and an allowance of 30% in excess of average working needs, yield a figure of 8.2 cars per route mile. Owing to uncertainty on this point at the time of estimating in July, 1917, a figure was used of approximately 9.3 cars per route mile or

Mr. Wilgus.

13 per equivalent locomotive, making the total number ordered 3 332, of the following types:

17-ft. bogie cars, open top, low sides	1 700
17-ft. bogie flats, with stanchions	480
8-ft. box cars	720
Bogie bolster	96
Tip cars	240
Bogie tank cars (1 500 gal. for army use)	96
	3 332

It was requested that the cars having a capacity of 9.3 long tons should have their contents capacity increased from 225 to 240 cu. ft., equivalent to the contents of the 10-ton standard gauge car of the European type, as practice had shown that this was highly desirable in transhipping ammunition at the rail-heads.

The cabled order, sent in July, 1917, also included 220 crossing-frogs of various angles, for intersecting standard-gauge and light-railway tracks; a selector telephone system for 330 miles of territory, with five dispatchers' outfits and station apparatus on the basis of stations one mile apart; 330 portable cabins; 48 portable gasoline pumps, including suction pipes and fixtures, each with a capacity of 3 000 gal. per hour against a head of 150 ft.; 48 000 lin. ft. of various sized pipe with valves and fittings; 48 demountable 15 000-gal. water tanks with valves and spouts; tools and machinery for a general repair shop and for 16 round-houses; maintenance tools for 420 track gangs; 20 locomotive cranes; a supply of 8 by 16-in. bridge stringers, and ample spare parts.

At that early date the question of forces for constructing, maintaining, and operating the light railway system was also considered. The experience of the British pointed to the following average needs per single track mile:

	Number of men.
Operation, exclusive of maintenance of way	10
Maintenance of way	14
Total	24
Construction (maximum)	20
Grand total	44

Therefore, it was contemplated that upward of 21 000 men would be required under these conditions.

In forecasting fuel consumption it was estimated that steam locomotives would require 48 lb. of coal per locomotive mile and gasoline locomotives 0.35 gal. per mile.

Mr. Wilgus.

The writer's connection with the "light railways" section of the Army's transportation service was terminated a few months after its inception, and hence he was not intimately in touch with its later development. He ventures the hope that the author will point out the faults of the original plan and procedure as here outlined, not only as an aid to those who later may have to cope with similar problems, but also as a warning against the inexcusable laxity and absence of ordinary foresight on the part of the War Department or other Governmental agency in having done absolutely nothing, prior to the entry of the United States into the war, to study the transportation needs and practices of the contending armies through experienced eyes, with a view to having in readiness a well thought-out plan embracing suitable provision for the material and personnel necessary for the prompt disembarking, storage, and distribution of American troops and supplies from the sea to the firing line.

It is also to be hoped that the author, in addition to pointing out the way in which possible errors in the early planning of the "light railways" may be avoided in future military operations, will also dwell on the extent to which the change from trench to open warfare modified the forecasts which were predicated on the preservation of the *status quo*. At the time of entry of the United States on the scene, the British considered that as far as any great advance was concerned, a forward movement of, for example, 5 to 8 miles in two days, would mean "open warfare", in which event the use of light railways would disappear; but it was of course unsafe not to order light railway materials on the theory that the Americans would be successful in immediately changing the character of operations. However, about the time the American Army commenced to function on a large scale, the enemy rapidly fell back on the entire Western front and this to a great extent relegated the light railways to the background. It will unquestionably be interesting and helpful to have known the extent to which this change in the military situation affected the usefulness of the means of distribution that had proven to be so indispensable while the armies were deadlocked.

Mr. Dunham.

H. F. Dunham,* M. Am. Soc. C. E.—The speaker wishes this paper had been extended to include facts relating to the traffic and operation of trains on so narrow a gauge. The author when concluding the discussion may mention difficulties that do not pertain to transportation on standard gauge lines.

Col. James B. Dillard, U. S. A., read a paper before the American Society of Mechanical Engineers, on December 5th, 1918, from which the following paragraph is quoted:

"The 1918-model, 8-in. gun, railway mount which uses the same barbette carriage as the 7-in. gun may be elevated from 0 to 42 degrees.

* New York City.

Mr. Dunham. It fires either a 200 or 300-lb. projectile at a maximum velocity of 2 000 ft. per sec. and has a maximum range of about 69 000 ft. By means of special 8-wheel trucks it is possible to transport this mount over a 60-cm. (24-in.) narrow-gauge track. This 8-in. mount weighs about 180 000 lb. and it certainly is a great advantage to be able to transfer it to a narrow-gauge track which is the kind used at the battle-front in order to better conceal operations. On the narrow-gauge track a speed up to 15 miles per hour is feasible. The Ordnance Department which developed this mount in its entirety considers this artillery piece to be the last word in railway mounts. It is also the only mount of its kind in existence."

Any further description of such unexpected loading and service would be especially interesting.

There are now many miles of 2-ft. gauge railroad in use in the United States, not as logging or factory-yard roads, but as veritable railroads serving many communities and functioning in all respects like a standard gauge steam road except that freight, passenger, and drawing-room cars are not exchanged, but always remain on their own tracks. Cars are not lost and a company can do its own repairing.

The Bridgeton and Saco River Railroad,* more than 20 miles in length, is a narrow-gauge (2-ft.) road of this type, which connects Bridgeton Junction on the Maine Central Railroad with Bridgeton and Harrison, Me. The alignment includes one 36° curve, one 20° curve, and others of longer radius. The grades reach a maximum of 4% for short distances. The road operates 5 locomotives, 6 freight and passenger cars, and a total of 72 cars. The locomotives weigh from 22 to 36 tons each. These engines have two driving axles about 54 in. apart, a one-axle pony truck and tender on the engine frame extended. The freight cars are more than 33 ft. in length, 6 ft. wide and 6 ft. high in the clear, weighing 7½ tons, and carry a net load of 15 tons. The noticeable difference in the passenger car is the "one-seat" feature on each side of the central aisle. The average speed is more than 20 miles per hour.

This road was built prior to 1888 and for many years before the World-War disturbance and high prices, it had paid regular dividends. In 1915 it carried 34 500 passengers and 31 500 tons of freight. These figures, however, do not show the great value of the road to those residents who without it would be shut off from the outside world for long periods. Its trains are said to be always waiting for the Maine Central trains in the intervals of trouble from deep snows; in fact, the narrow gauge is said to be an advantage in heavy drifts.

From Farmington, Me., in a northerly direction, the Sandy River and Rangeley Lakes Railroad of 2-ft. gauge has now 106 miles of similar roadbed and track and 16 miles of sidings. This road operates 14

* The speaker illustrated his remarks by a moving picture film showing locomotives, cars, and roadbed of this railroad.

locomotives, 18 passenger cars, and a total of over 300 cars, and the revenues from operation in 1918 were $234 955. The weight of rail varies from 25 to 58 lb. per yd. Mr. Dunham.

A traveler on these narrow-gauge roads is not subjected to hardship, and records do not indicate added risks. Extra fare insures a seat in the drawing-room or chair car and the principal difference in the passenger's sensations is a feeling that the ground is not far away. The low center of gravity makes for safety. When cheaper motive power, for example, hydro-electric and storage battery service, can be provided, such 2-ft. gauge lines should become more numerous where there is not business enough to justify standard gauge construction.

Morris E. Pumphrey,* Esq.—The speaker was with the 21st Engineers at Sorcy, Meuse, which place has been described by the author, from March to October, 1918. Afterward, he was in the Argonne, and it is thought that a few details of his experiences may be of interest. Mr. Pumphrey.

One of his first duties on arrival at the front was to make a reconnaissance for the purpose of connecting the light railway lines in the St. Mihiel Sector with the German lines, in case of advance. This plan was properly studied and prior to the St. Mihiel advance, large quantities of light railway material were brought up adjacent to the front-line trenches. On the morning of September 12th, 1918, preparations had been completed for the laying of the tracks across "No Man's Land", as soon as the infantry had advanced.

The speaker was located at a town called Flirey, about 400 yd. from the front-line trenches, where he had one company of experienced light railway troops, with one company of negro labor troops, which had just arrived in France and had had no experience with construction work. These were the only troops available for this work and were totally insufficient in number. On the morning of September 12th, the infantry went over the top about 5 o'clock and the engineering troops immediately started to construct the light railway across "No Man's Land". As the infantry advanced, a party of experienced engineers laid out the line on the ground and marked it with the white trench tape. As may be imagined, this line was very uneven and lacked a great deal in the way of uniform grades and curves. At this point 4 km. of track had to be built. Due to the shortage of labor and lack of ballast, it required 4 days to get this line in operation, whereas it should have been built in 12 hours in order to have been of real use to the combatant troops.

After this line was completed, the regiment to which the speaker was assigned was moved to the Argonne Sector, where no light railway troops had been available before. The Argonne advance began on September 26th, 1918, but it was almost the middle of October before

* New York City.

Mr. Pumphrey.

any definite headway had been made in connecting the American light railways with the German lines. This was due to an insufficient number of troops, lack of material, and a great lack of proper planning before this operation was put into effect. By November 1st, the light railways in this sector had been well connected up and traffic was operating smoothly. As the combatant troops advanced November 1st, for the final phase of the Argonne battle, more light railway troops and material had been made available, so that for a few days it was possible to maintain connections with the advancing infantry, but by November 11th, the infantry had advanced so far that the line of supplies had almost collapsed. On November 11th, supplies were being hauled on one line for a distance of 55 km. and on another for a distance of 60 km. Both these lines were handling approximately 1 000 tons per day of rations and ammunition.

The locomotives which were furnished for light railway work proved to be very unsatisfactory, the steam locomotive being top-heavy. The speaker recalls the case of one Captain in the regiment, who re-railed 65 of these locomotives in 1 month, which would seem to be a record. The gasoline tractors worked very well at times, but generally failed at the crucial moment.

It was found that curves with a radius of 50 m. are as sharp as should be used and that no grades exceeding 3% should be considered. Recently, in one of the engineering papers, some writer has suggested the advisability of adopting wooden ties, instead of steel ties for sectional track such as was used by the Armies engaged at the front. It was quite right to use the wooden ties on back lines where there was ample time for the construction of first-class track, but it certainly would not be advisable to use wooden ties near the front, on account of the delay in transporting them.

The speaker is firmly of the opinion that the 20-lb. rail used by the English is the ideal rail for the 60-cm. railways. However, it is understood that the engineers of the War Department are recommending a 25-lb. rail.

During the early part of 1918 the speaker made a detailed study of the French light railway system from St. Mihiel to St. Die in the Vosges Mountains and found that sharp curvature and heavy grades were common on the French railways, that the road-bed as a rule was too narrow, and that insufficient ballast was provided to make a good track, although in numerous cases masonry retaining walls and culverts which would be a credit to the standard railways in the United States were found. This work, as can readily be imagined, was absolutely unnecessary in war-time construction.

Mr. Hoyt.

HAZEN L. HOYT, JR.,* ESQ.—If the American people are to derive any benefits from the World War, it must be by profiting by the

* New York City.

Mr. Hoyt.

lessons learned both at home and abroad. It is necessary, therefore, that the good and bad points of the different services and departments be carefully weighed before drawing conclusions, and this is true of the light railways of the battle fronts in France.

The author has given a clear idea of the types of equipment used abroad and of the work done by the light railways in France, but he has failed to bring out wherein they failed in their functions.

At the outbreak of the war many kilometers of military railways existed in France, principally in and around the permanent fortifications of that country, and it is, therefore, only natural that when the fighting settled down to what was known as a "warfare of position" that these railways should be extended to help supply the troops in the line, especially as motor-truck transportation has never been developed abroad to the extent that it has in the United States. However, the Allies soon realized the need of this form of transportation, and large orders were placed both at home and abroad to supply this demand.

When the United States entered the war these 60-cm. railway lines were well established, and portions of them were taken over by the Americans with the sectors assigned to them. The railway locomotives used by the Americans in France were manufactured by the Baldwin Locomotive Works, as stated by the author, but, in the speaker's opinion, proved to be absolutely unsuited to the service they were called on to perform. The steam locomotives were entirely too high, had insufficient steam capacity, and were badly adapted to the kind of coal they had to use. The spring suspension was poor and only 68% of the weight of the locomotives was carried on the drivers. The over-all length was too great in proportion to the rigid wheel base, causing the rear drivers to pull off the rails when hauling heavy trains around sharp curves. Although these locomotives were designed to negotiate 30-m. curves, it was found that 40-m. curves had to be adopted as the minimum and curves of 50 m. were used where possible. Why this type of locomotive should have been adopted, the speaker is at a loss to understand, as double-truck steam locomotives had been manufactured in this country for the Allied Governments prior to the entry into the war by the United States and had proved entirely satisfactory. So poor was the ability of these locomotives to stay on the track, that one company of the 21st Engineers operating and maintaining approximately 50 km. of track during the Argonne Drive had to re-rail 65 locomotives in 1 month.

The gasoline locomotives worked better as far as their ability to stay on the track was concerned, but the standard of workmanship and material was low, and mechanical troubles were constantly arising, principally with the clutch and gear case. The spring suspension also was poor which, combined with the large overhung weight

Mr. Hoyt. at either end, caused a jumping effect at high speed, frequently resulting in derailment.

The cars were far too heavy for their load capacity and trouble was constantly being experienced with the draw-bars, due largely to the great amount of side play allowed, resulting in frequent instances of broken draw-bars when coupling up the trains. If a four-wheel car of half the capacity had been adopted, the weight could have been reduced by nearly two-thirds, besides such cars would have been far easier to re-rail than the eight-wheel double-truck type furnished.

The author states that both the English and French Armies had found that it took approximately 21 000 man-hours to build 1 mile of 60-cm. track. In this connection, it would be well to give some figures of the work done by the 11th Engineers during the Cambrai Drive in November, 1917. Working in conjunction with about 350 men of the 4th Battalion of Canadian Railway Engineers, 300 men of the 11th Engineers laid approximately 10 miles of standard-gauge track in 7½ days, from Epehy to Marcoign. The best time was made by the Americans, who laid 1¼ miles of track in 5 hours and 46 min. This work consisted in re-laying the rails and ties, filling in shell holes, mine craters, blown-out culverts, trenches, etc., the ballast being already in place as the line followed along the old right of way of the Nord Railroad, running from Peronne to Cambrai.

Another matter which caused endless trouble for the troops operating the light railways, was the voluminous reports required by the Department of Light Railways, many of which it was impossible to supply and almost all of which could not be given accurately, due to the conditions of operation. This was particularly true of the reports on coal and gasoline consumed, as much of this material was obtained from captured German dumps and the quantity could only be approximated, as no weighing apparatus was available.

Another source of annoyance was the difficulty, if not impossibility, of securing permission to change the equipment in any way, as this meant a change in the standards. Many of these changes were minor, but would have helped materially in the operation of the trains and could have been done easily by the troops themselves had permission been granted.

As stated by the author, the main shops of the light railways were located at Abainville, almost directly back of the center of the Toul Sector; this town also was the supply base for the light railways. Until the lines started to advance it served its purpose very well, or would have, had the shops been completed. After the First Army moved to the Argonne, however, this base was about 70 to 80 km. away by road and nearly twice that distance by rail, requiring nearly three days' travel by rail and almost a day by road in order to obtain supplies. Even when these supplies arrived at the light railway shops of the First Army, at

Dombasle (in Argonne), they were nearly 60 km. from the advanced railway companies. Mr. Hoyt.

These were the conditions existing at the time of the advance of the American troops in the Argonne, and previously on a lesser scale in the St. Mihiel Salient. The first proposition that confronted the Americans, of course, was to throw a line across "No Man's Land" in order to connect up with the abandoned German track. Prior to the opening of the St. Mihiel offensive a careful study had been made of the situation, and material of all sorts had been provided for connecting up the German lines at three points: From Flirey to Essey, from Bois Chenau to Woinville, and from Point au Metz to St. Baussant, the average distance being about five miles, including the approaches on both sides and the distance across "No Man's Land". Work was started simultaneously on all three lines on the morning of September 12th, 1918, but it was not until about October 1st that these lines were fit for train operation, in spite of ample troops and material having been provided beforehand.

This same condition existed in the Argonne, aggravated by a marked shortage of rails, ties, etc., as time and transportation had not been available for their concentration back of the lines. So great had been this delay that at the time the Armistice was signed the farthest point of light railway operation was about Buzancy, or approximately half way from Varennes to Sedan. Furthermore, this point marked about the end of the system of German light railway track abandoned by them in their retreat, and from here the Americans would have been compelled to build entirely new lines to supply the troops if the forces had been pushed ahead. This condition existed in spite of the fact that somewhat more than 6 000 men were employed on the light railways of the First Army at that time.

From this it can be readily seen that, although the light railways had proven their value in a "war of position", they proved totally inadequate in following up an extended advance, and the troops were forced to fall back on the road transportation for their supplies.

Reviewing the conditions as they existed on the St. Mihiel and Argonne fronts during the advance of the American forces in September, October, and November, 1918, it is the speaker's opinion that better results would have been obtained if every effort had been centered on pushing forward the standard-gauge rail-heads, which at the time of the Armistice had reached Varennes, and improving the roads, so that the greatest possible volume of road transportation could have been used for the supply of the troops. This would have necessitated a careful study of the available roads and a well-trained body of Military Police to handle the traffic and avoid congestion. In order that these roads might be kept in the best possible shape, the men and ballast used on the light railways should have been diverted for this

Mr. Hoyt. purpose, leaving the 60-cm. lines to handle the salvage collected from the battlefields, pushing forward their lines as conditions permitted.

Mr. Poland. W. B. Poland,* M. Am. Soc. C. E.—Illustrating the critical situation as to transportation at the front: In the early months of 1918 the speaker attended a conference between representatives of the American Expeditionary Forces and the British Army Transport Service, represented by Gen. Thornton, formerly of the Long Island Railroad, then in charge of transportation along the British front.

After explaining the terrible straits of the Allies, for cars and locomotives, Gen. Thornton said: "If the line stays where it is, we can probably get up our supplies and ammunition, but if the Boche ever retreats, God help us, we're lost!"

Mr. Pugh. Marshall R. Pugh,† M. Am. Soc. C. E. (by letter).—The author has touched but briefly on a phase of light railway work about which he is well qualified to speak, namely, the light railways in the extreme forward areas, where conditions differed materially from those obtaining farther away from the trenches.

The light railways as operated by the American Expeditionary Forces may be divided into three zones:

(*a*) Near the rail-head, operated by steam locomotives.

(*b*) The forward area, operated by gasoline locomotives.

(*c*) Tram-lines at the extreme front, using animal or man power.

Wherever the lines were far enough from the enemy to be reasonably safe from shelling, steam locomotives were used, as they were more powerful and could haul heavier trains. The gauge of track was 1 ft. 11½ in., and the over-all width of the steam locomotives was 6 ft. 5 in., consequently they needed a substantial track. Nearer the front, the light at night and the steam and smoke by day made them undesirable, so gasoline locomotives were used. Where their use was impracticable even at night, as well as on numerous spurs to batteries, mules and horses were used, or the ammunition and supplies were transferred to light cars pushed by men. Fig. 10 shows a light car transporting water on a spur reaching within a few yards of the front-line trenches, and is typical of tram-line conditions at the extreme front.

The first, or (*a*) zone, approximates more nearly to an ordinary railroad, and has wooden ties, rock or gravel ballast, and comparatively light grades and curvature. The road is divided into blocks with a telephone train-despatching system. Where not subject to shelling, small huts were used by the despatchers, as shown in Fig. 11. These huts were made in sections hooked together and could be taken down in a few minutes, moved, and re-erected at another place with equal rapidity.

* New York City.

† Wayne, Pa.

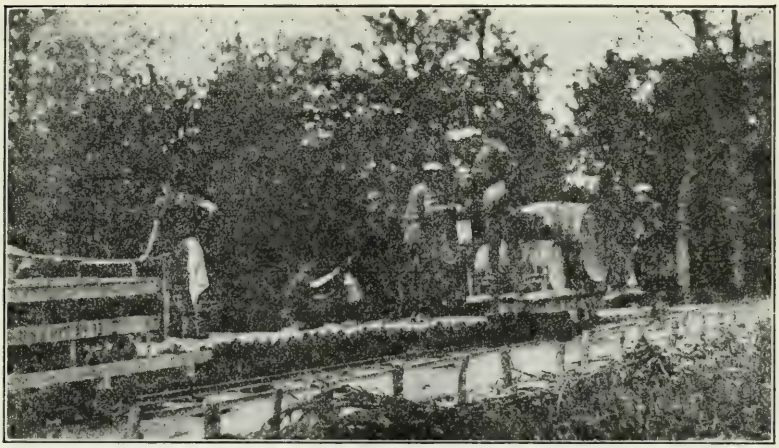


Fig. 10.—Water Car on Tram Line.

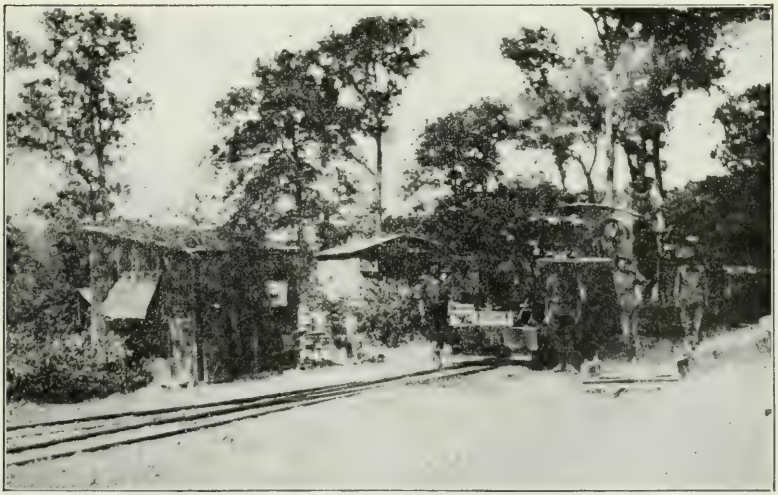


Fig. 11.—Portable Despatcher's Hut, La Reine.

Fig. 12.—Despatcher's Hut, Broussey, Showing Sand-bag Protection.

Fig. 13.—Camouflage at the Front.

Mr. Pugh.

In the forward zone semi-circular steel elephant huts were used to house the despatchers. These huts were protected by sand-bags as shown in Fig. 12, which is a view of the despatcher's hut in Broussey. This hut was at the terminus of the gasoline-operated section, beyond which animal traction was used. All operation was by night, and the curtain screen hung across the roadway was to hide it from enemy observation.

The road in exposed places was screened by camouflage, as shown in Fig. 13. At first, there was a great deal of hesitancy about building much in the open in forward areas, but experience showed that it is very difficult to hit such a narrow target unless it is directly enfiladed. Near Beaumont, a point called "Dead Man's Curve" was subjected to shelling for months, but comparatively infrequent repairs to the track were necessary. When a shell did hit the track, it was a simple matter to take out the damaged section, or sections, as shown in Fig. 14, and replace them by new sections. Wrecking trains were stationed at intervals, on which were carried some full length (5-m.) sections of track and some half and quarter sections, ready to make a quick run to any damaged point and replace the track. The difficulty of hitting the track and the ease of repair makes it far more costly to destroy a light railway by shell fire than to restore it. It is about as impracticable to destroy such a railway by shell fire as it is to get rid of cooties in the same manner. This fact is their protection.

There were two abandoned meter-gauge railways in the Toul Sector, and in places much work and ballast was saved by laying the light railway track in the center of the old meter gauge, as shown in Fig. 15. The dug-outs and huts shown are in the shelter of the old railroad cut just behind the trenches and are occupied by the troops holding that part of the line. As illustrating the amenities of life in this region, one of the writer's corps making a railroad location in "Shrapnel Valley" just to the right of this point, was hailed one day by an officer who said that the Boche had broken through and he wanted the survey party to help man the machine guns till reinforcements arrived. The survey was temporarily deferred.

Just south of this point the light railway left the meter gauge, which formerly crossed on the bridge shown in Fig. 16. Barbed wire entanglements are shown in the foreground, also one of the narrow-gauge "speeders". These little speeders were used by officers and men, repair gangs, and the like, for getting over the road, and were indispensable. Their usefulness would have been far greater, however, had they been adapted to the service they were called on to perform. There were two types. The one shown in Fig. 16 held six persons; it had one cylinder, was cranked by pushing it along the track and, when on a heavy grade, good track was absolutely essential, as it would stall if not moving rapidly. The smaller speeder, holding three persons, had two cylinders, was swifter and more easy riding, but stalled even more

Mr. Pugh. readily on grades and jumped the track on no provocation whatever. The writer preferred it to the other, but took many an unpleasant spill. Derailments were constant with each type, as they had no clutches, and although they would run well on a smooth track, they could not be operated slowly and were entirely unsuitable for conditions at the front. Several serious injuries and one death occurred from their use by the 21st Engineers. They should have had clutches, means of cranking, and have been geared so that they could run slowly on poor track. There is a good, light friction-drive speeder in the market.

The general instructions on location were to avoid heavy work, and Figs. 11, 13, 14, and 18 indicate the character of the great bulk of the line constructed. Where the track leaves the rail-head at Sorcy Yard, on the curve marked "To Cornieville" on Plate XXVII, there was a ridge to be crossed, and as heavy traffic had to be handled, the writer put in the only heavy work in the line—a 17-ft. rock cut shown in Fig. 17. A French highway is carried by the bridge in the foreground. There was considerable criticism prior to the completion of this cut, but the writer heard of none after it was finished and operation showed the justification for it. In general, however, work of this character would be utterly misplaced in light railway lines. Ease and speed of construction are generally of paramount importance, and it is also necessary to know just how long it will take to construct a given line under prescribed conditions as to number of men available. The construction forces must be divided in such a manner that grading, track-laying, and ballasting will progress with equal rapidity, so that the line will be completed without any delay due to the track being held back by grading, or by ballast not keeping up with the track. Conditions varied materially, but a rough average of the lines in the forward part of the Toul-St. Mihiel Sector indicated that with approximately a 1-ft. cut or fill, the forces should be proportioned as follows:

Clearing	5 men.
Grading	60 "
Track-laying	10 "
Ballasting	25 "

Progress charts were prepared, and when the work was completed, it was checked against the preliminary estimates, as shown in Table 2.

The Sorcy-Cornieville Line aggregated 2 344 man-days per mile, comparing in this respect with the Abainville Line cited by Col. Jonah. Each was what may be termed a trunk line. The Hamonville and Raulecourt Lines, on the contrary, were close to the front, with light grading and sectional track. The Hamonville Line averaged 1 024 man-days per mile and the Raulecourt Line 1 169 man-days per mile. The Abainville Line and the southern part of the Cornieville Line were laid with wooden ties. At the Cornieville end steel ties and sectional track

Fig. 14.—Replacing Track Damaged by Shell Fire.

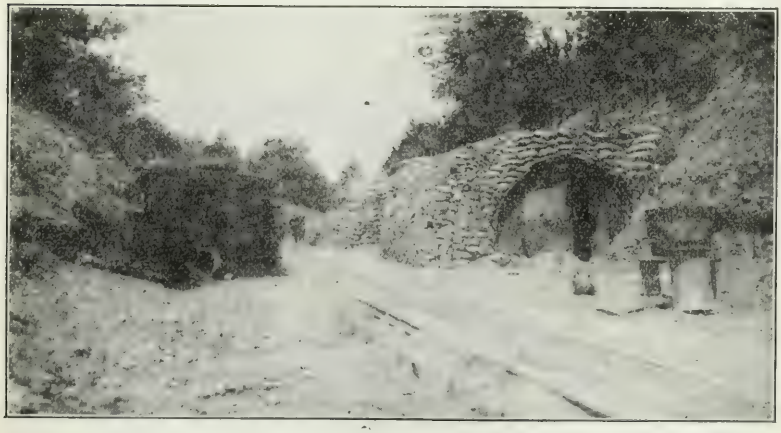


Fig. 15.—60-Cm. Track Laid on Old Meter Gauge Line.

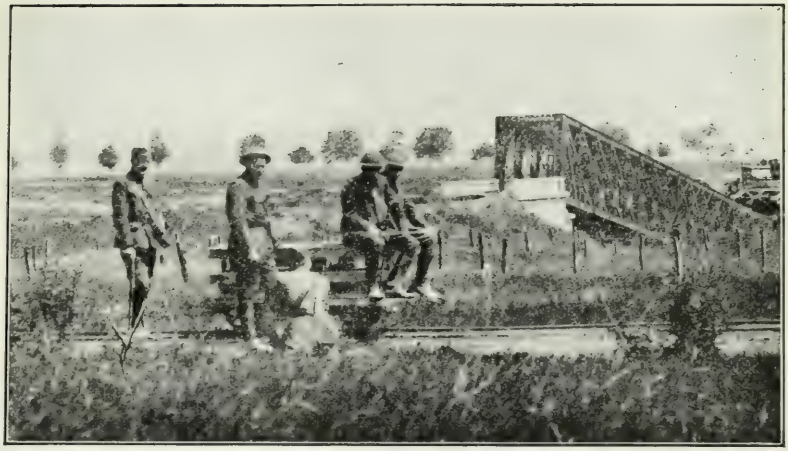


FIG. 16.—LIGHT RAILWAY "SPEEDER", ENTANGLEMENTS AND DESTROYED BRIDGE NEAR FLIREY.

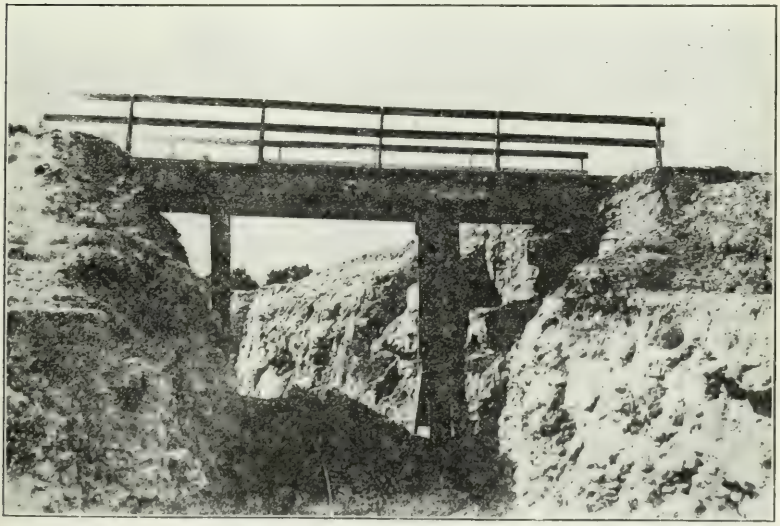


FIG. 17.—EXCEPTIONAL WORK ON LIGHT RAILWAY; 17-FT. ROCK CUT AT SORCY.

were used, which afterward were replaced by wood. To avoid interruption to traffic during the change, the wooden ties were placed between the steel ones, and the new rail, bolted together in sections several hundred feet long, was laid on either side of the sectional rail. After the passage of a train the sectional track was lifted out, thrown to one side, and the new rail quickly spiked to the wooden ties, only four bolted connections being necessary for the entire section. Fig. 18 shows this operation. The new rail is ready to be spiked into place as soon as the sectional track is lifted. In the foreground enough of the 25-lb. American rail on wooden ties is shown to indicate its superior alignment and solidity. Mr. Pugh.

TABLE 2.—Estimated and Actual Percentages of Work on "K" Line (Raulecourt).

	Estimated percentage.	Man-days.	Actual percentage.
Clearing	4	69	5.11
Grading	60	751	55.59
Track-laying	10	199	14.73
Ballasting	26	332	24.50

It will be noted that the lines at the extreme front averaged only about half as many man-days per mile for construction as those farther back. They had few bridges and all the work was light. On the contrary, given an equal amount of traffic, the maintenance on these lines was greater, and wooden ties held up much better than steel ties.

What lesson experience in the World War teaches as to the defects that developed in the light railways, and the remedy, is a question of utmost importance.

Paul McGeehan, M. Am. Soc. C. E., in an article entitled "Notes on Army's Light-Railway Practice in France",* has put his finger on what the writer considers the lesson to be profited by if light railways are to be made the success which their importance demands. Capt. McGeehan says, "The number of derailments was almost a scandal. On one classic occasion at Tincourt, on the British front, all of the steam locomotives in the service were on the ground at the same time. Derailments were so numerous and the cause of so much loss of time that orders had to be issued to reduce speed to not more than eight miles an hour."

The first and most obvious part to which consideration should be given is the roadbed. Ballast is by all odds the most serious problem in light railway construction. In Table 2 it will be noted that 24½% of the labor, on that particular branch, was used on ballasting, but this does not begin to cover what is involved in that part of the construction.

* *Engineering News-Record*, October 2d, 1919.

Mr. Pugh. The quantity of ballast needed and the long hauls frequently necessary before it can be placed, choke the railroad with ballast trains. In the week ending April 19th, 1918, of 4 674 tons hauled on the light railway, 1 290 tons consisted of ballast. For the week ending August 31st, 1918, the total hauled was 19 885 tons, of which 2 717 tons were ballast. The following week the figures were 34 942 tons total, 7 954 tons of which were ballast. In general, the ballast varied between one-tenth and one-quarter of the tonnage moved. Capt. McGeehan calls attention to one instance where ballast had to be fed continuously into certain lines in order to keep ration trains running. "Ballast, even, had to be put under this track to keep the ballast trains themselves running." He agrees with Col. Jonah in considering wooden ties preferable, because surface and line are more readily kept, and steel ties cut into the ballast. Their contention is not open to dispute, and when it can be done the writer would advocate their utilization, but he feels that in the forward areas the superior speed with which sectional track can be laid, especially at night, the lessened demand for skilled labor, and the avoidance of noise due to spiking, make the bolted sections with steel ties vastly preferable.

Much of the sectional track used by the American forces was French Decauville track with 16-lb. rails. This is too light for main track, and the head of the rail is too narrow to give the drivers proper adhesion. The American 20-lb. and 25-lb. rails were superior in every way. Personally, the writer prefers the 20-lb. rail, as it seemed to be quite as satisfactory and is lighter to handle. It is a mistake to have both. The difference in appearance between the 20-lb. and 25-lb. sections is negligible, and in one instance led to disastrous results. A start was made to lay several miles of sectional track, and after quite a stretch was down, the sections failed to fit. It was then discovered that the erection shop had forwarded to the front sections having one 20-lb. and one 25-lb. rail to each section. It was not noticed as long as the sections happened to have all the 25-lb. rail on the same side. When, however, a 20-lb. rail came to match up against a 25-lb. rail, trouble began, and a gentle critique of the erection shop by the track-layers ensued.

The American standard sectional track, 5 m. (16 ft. 5 in.) long, had eight ties, with a total tie area in bearing of 13.76 sq. ft. per section. The German sections were not all alike, but in general had ten ties to the section. One excellent type had a total bearing area of 23.2 sq. ft., or 176% of the area of the American section. The American 20-lb. rail section weighs 406.4 lb. The German section with American 20-lb. rail would weigh 549 lb. Although heavier to handle, it would not be objectionable. Furthermore, the German ties are 4 ft. long, 5 in. longer than American ties, giving somewhat superior stability. Of most importance, however, is their width which is 7 in. as against 5½ in. for

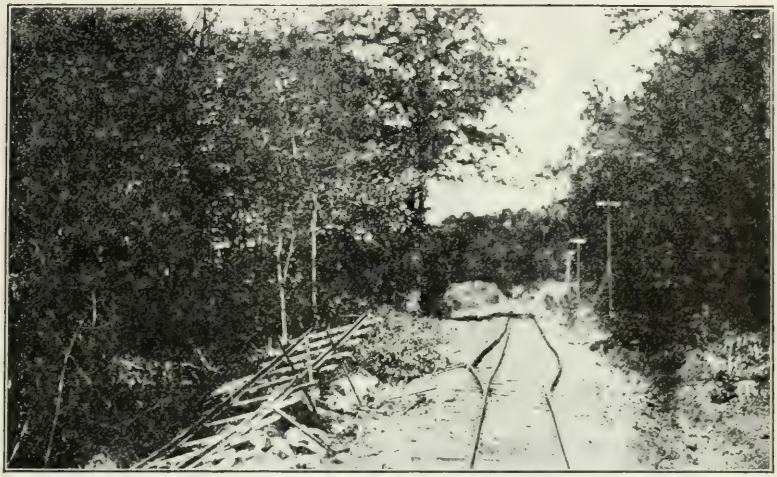


FIG. 18.—REPLACING SECTIONAL TRACK BY RAIL WITH WOODEN TIES.

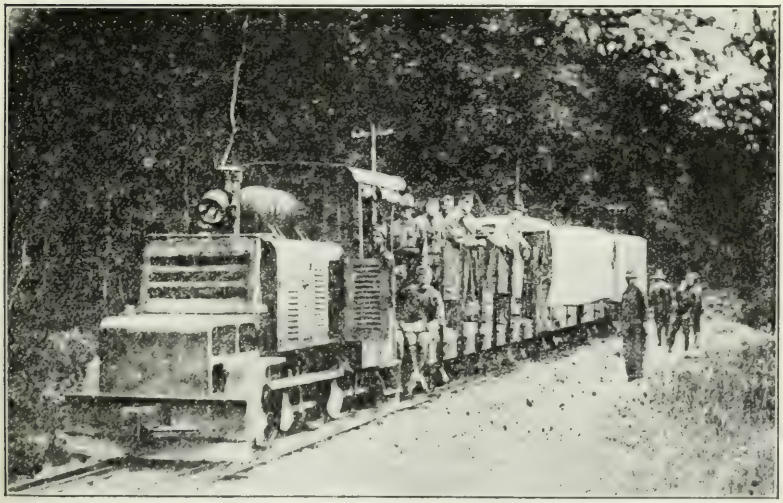


FIG. 19.—LIGHT RAILWAY GASOLINE TRACTOR AND FLAT CARS.

American ties. The Second Progress Report* of the Special Committee to Report on Stresses in Railroad Track throws an interesting light on the relative supporting capacities of these two widths of tie. It shows that the supporting capacity is greatly increased by increase of width. Table 18 of that report indicates that "with jarring" the German tie would support a load of approximately 160% of that of the American tie. Since there are 25% more ties to a section, the superiority of the German section on a poor soil is apparent. Mr. Pugh.

The writer would advocate the continued use of sectional track at the extreme front, or wherever speed in construction is essential, using on soft ground a mat of brush, leaves, or the like, when obtainable, before placing the ballast. Where wood is available, he has found great benefit to result from inserting between the steel ties on bad ground either wooden ties, planks, or boughs of trees, to give added bearing power and stability.

As to gauge, where an enemy's lines have the 60-cm. gauge, as was the case in France, it is undoubtedly preferable to adopt the same, so that his lines may be utilized in an advance. Otherwise a 30-in. gauge would be preferable. Neither the rolling stock nor the track would weigh any more, and the tendency to derailment would be less.

Practically all the articles which the writer has read on the subject regard the great part of the troubles in light railways as arising from poor roadbed. Desirable as some track changes may be, they are of minor importance, in the writer's judgment. It must be borne in mind that under battle conditions the roadbed must and will be poor. It is vitally necessary to get cars and engines that will run on a poor roadbed. This has been the writer's contention from the start. American rolling stock was based on the idea of a miniature standard-gauge railway. It should be based on that of a contractor's rough-and-ready construction railway. An interesting side light is thrown on this by a remark on page 218 of the "Historical Report of the Chief Engineer * * * American Expeditionary Forces," where it is stated: "It was found that the lines of the German system contained many sharp curves and heavy grades, which called for reduction before American equipment could be successfully operated." Lieut. Harvey W. Bell, of New York City, formerly of the 21st Engineers, A. E. F., a mechanical engineer of great experience in industrial railways prior to the war, as well as in operation of light railways in France, writes:

"I took six trains of about seven cars each, using steam power, from Sorcy to the Argonne. We passed near Mt. Sec and ran near the new front over German track and had some pretty exciting times. Our locomotives made so much noise and showed so much light that they must have been easily located. To add to our troubles both engines and cars seemed always to leave the track as soon as shells began to land near us.

* *Proceedings*, Am. Soc. C. E., February, 1920, p. 133.

Mr. Pugh. One night in the Argonne our steam engine turned over on its side on a bad curve, blocking the main line. We worked until midnight getting it up again, and no sooner were we prepared to start than it turned over on its other side and we didn't get it up till morning. I could tell you a hundred stories of a similar nature with our equipment, yet I never saw a French engine or gasoline locomotive off the rail."

The American gondola and flat cars were 24 ft. 1¼ in. long over couplers, and 15 ft. center to center of trucks. The weight of the gondola was 9 000 lb. and of the flat car 8 000 lb. Lieut. Otis T. Gregg, of the 21st Engineers, also a man of wide experience in industrial railways, suggests a four-wheel car which could be made of one-half the capacity and one-third the weight and which six men could lift on the track. The little four-wheel, V, dump cars, 6 ft. 9 in. long over couplers, kept the rails on rough track and would go around curves which the other cars would not begin to take. Fig. 21 shows how heavy loads can be moved by very simple rolling stock.

The steam locomotives (Fig. 11) were too high, had a bad spring suspension, and only 68% of the weight of the locomotives was carried on the drivers. The rear drivers would pull off the rail on the inside of the curve when hauling a heavy train over sharp curves, because the over-all length was too great in proportion to the wheel base.

In locations fixed by the 21st Engineers compensation for curvature on grades was made at the rate of 0.02% per degree of curve. Nevertheless, with the long cars operated by the Americans, curves offered much greater resistance to traffic than grades.

The writer's regiment was fortunate in having several mechanical experts of great ability, and they were in entire agreement with him—he is not a mechanical engineer—as to the desirability of a different type of rolling stock. Lieut. Alfred D. Chandler, Jr., now representative of the Baldwin Locomotive Works in South America, was most insistent on the need for a revision in the design of American engines. A double-truck gasoline locomotive with all the weight on the drivers, a motor of about 75 h. p., and a four-speed transmission, to replace both the steam and the gasoline locomotives would be, in the opinion of those better qualified than the writer to pass judgment, a great change for the better. In France, conditions occurred in which the locomotives had to remain miles away from proper supplies of coal and water and pit tracks. Gasoline locomotives could carry supplies of all kinds sufficient for four days' hard work.

In stressing the question of rolling stock, it is possible that the writer's meaning may be misconstrued. It is not his intention to advocate poor track. Make it the best that is possible, but when conditions compel that "best" to be poor, have cars and engines which will run on the worst of it, and which can readily be re-railed.

FIG. 20.—FLAT CAR LOADED WITH AMMUNITION.

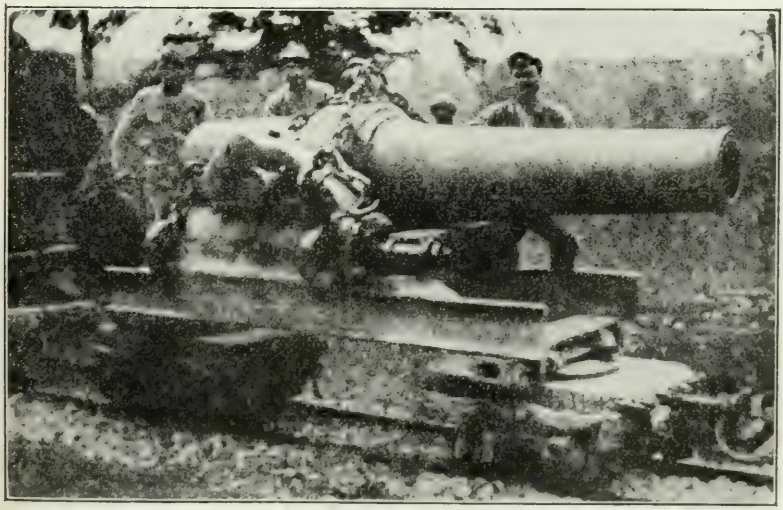


FIG. 21.—HOWITZER ON THE LIGHT RAILWAY.

Mr. Pugh

The light railway is not adapted and should not be used for long hauls, which can be far better taken care of by the standard gauge. It is not practicable to destroy it by shell fire, so that it should be used where the standard gauge cannot be operated, with necessary extensions to the rear to reach properly located shops and to enable the withdrawal of equipment in event of an enemy advance, these extensions to be located and built in accordance with the best principles of standard-gauge practice.

On page 1221, Col. Jonah has clearly and succinctly stated the primary reasons for the development of light railways. Some concrete examples of what the light railway does may be of interest. Maj. G. Boyer, of the 4th Canadian Division, in a report to Sir Henry Jacob, gives an idea of the great saving in work effected by them. He says:

"Before the tram lines were organized and constructed one horse and one man brought to his battery eight 18-lb. shells, therefore to deliver 16 000 rounds it took 2 000 horses and 2 000 men; by the tram lines, taking as an average a hundred rounds per truck, it would take 160 horses or mules and 184 men; saving and allowing to rest 1 816 men and 1 840 horses or mules."

Light railways are also of the greatest aid in troop movements, saving tiresome marches (generally at night) and relieving the already congested highways. An extract from a citation to the 21st Engineers by the Commanding General of the 82d Division illustrates this point very clearly. He says, in part:

"The light railway is a great saving on trucks and gasoline; also a great saving on the men, who were saved physical exertion moving about and were able to apply the exertion in other more important work. It also made it possible for the men to move quickly during the reliefs, where to camp men in unusual stations or move them about in daylight would attract enemy fire and suspicion. Some 20 000 troops were moved in a recent relief, much to the benefit of the troops."

The writer would like to take this opportunity of expressing his appreciation of the great help which the author's keen observation and trained judgment was to him on the location and construction of the light railways. He is deeply indebted to Col. Jonah for his criticisms and suggestions.

Mr. Perry.

Francis W. Perry,* Assoc. M. Am. Soc. C. E.—The speaker's service with the American Expeditionary Forces in France was first as Quartermaster, then as Deputy Chief of Staff of a combat division (77th Division), and, finally, as a member of the General Staff of the First Army. The division was attached to the British Expeditionary Forces in April, after the opening phase of the 1918 German offensive, on the right of the British front, in Picardy, and during the second phase, on the left, where the offensive was directed toward Hazebrouck,

* Brooklyn, N. Y.

Mr. Perry. in the Hazebrouck area. During the third phase (on the left of the French front, toward Paris) the battalions of one infantry brigade (the 154th) were put in the line on the new Picardy front. Then the division was moved to the Lorraine front, where it was brigaded with the French, at Baccarat. The headquarters of the First American Army were formed near Chateau Thierry, at La Ferte-sous-Jouarre, in July, 1918, and the speaker joined in the same month, serving in turn as Secretary of G-3 (Operations) and as Chief of Sub-Section, through the "Second Battle of the Marne" and through the preparations for the reduction of the St. Mihiel salient.

Two types of narrow-gauge field railways were in actual use on the different fronts. The French used both the 60-cm. and 40-cm. gauge track, but the difference between the two types was one of character rather than gauge. The British distinguished them by calling one type "light railways" and the other "tramways".

The French staff maps show the many meter-gauge railways of Belgium and France as "tramways". There will be no confusion of terms when it is remembered that these tramways are privately owned roads and the narrow-gauge field railways under discussion are purely combat railways.

The so-called tramways were the field railways of the forward areas, beyond reach by other railways. They were built of track of the portable type. Tram locations were made with more cunning, in woods, folds of the ground, or in trenches, and with less regard to limits of grade or curve, than were necessary for the light railways. They were operated by light tractor, gravity, horse or mule, or man power.

As a result of experience with movements of the battle line, there was a tendency to build the main forward tram lines of 60-cm. gauge tracks and of the same weight of rail as that used for the light railway, and to locate them with a view to pushing them forward, following an advance, and then developing them as light railway lines of connection with the enemy systems of field railways. The ordinary function of these tramways was to supplement the regimental horsed transport, and pack mules and carrying parties in the trench areas.

The British had used a 9-lb. tram rail, but afterward this lighter weight rail was reserved for short lines, as between artillery dumps and battery positions. It was not necessary that the tramways, as lines of supply distribution in the trench areas, should take off from the light railways; they were as useful from the railway or road dumps.

Referring to the map (Plate XXVI), the standard-gauge railway—Conflans-Baroncourt-Longuyon-Montmedy—circling the Briey iron district, was the main supply line of the Army of the West. This line was a highly improved, double-tracked road paralleling the front from

Switzerland to the North Sea—a *voie-de-rocade*. In general, the narrow-gauge field railways were located in the zone between the main supply line and the battle line, on all the fronts. Mr. Perry.

Viewing the front from the German side, the strong Verdun salient threatened the main supply line. The Germans had one other way around, namely, the standard-gauge railway north by way of Thionville to Luxembourg, where connection was made with the Coblenz-Luxembourg-Namur-Brussels line. These supply lines were cross-connected, notably at Longuyon, an important junction with many long sidings.

The German right on the Verdun salient was the American Meuse-Argonne front. Here, the field railways appear as typical area systems. That on the extreme right was located in and about the Argonne Forest. At the close of the war, the German light railways had reached a depth of about 20 miles from the organized front. They paralleled standard-gauge railways forward from about that depth which were, in many places, combat railways of that gauge. They also paralleled good roads and, frequently, canals.

A function of the Services of Supply, or the assimilated organization, was to bring up the supplies to points within convenient reach of the troops. On the Western Front, standard-gauge and meter-gauge railways, navigable waterways, and roads were used for the bulk shipments, and the rail, water, or road-heads were as far forward at any time as they could be worked. Early in the war these were generally nearer the front than they were later, when heavy artillery was commonly used. The most important instance of long-range shelling by mobile guns was the shelling of points in Longuyon and elsewhere on the German main supply line in the closing fortnight of the war, carried out by the American railway naval batteries from the tracks in the northerly environs of Verdun. The ranges exceeded 20 miles.

The forward areas at the front, where the tramways were used, comprised a belt from 2 to 3 miles in depth on either side of the battle line. They were under intensive enemy observation, to the smallest detail of visible activity or the least sound that could be heard, and were more or less devastated, depending on the liveliness of the sector. They were occupied by the infantry divisions in the line, which also operated the tramways. The devastated area, including the zone between the organized front lines ("No Man's Land"), averaged about 5 miles in depth, and that was about the average distance between the light railway forward dumps of the opposing sides. Following an advance, the light railway troops pushed the tram lines forward with great rapidity and at the same time they brought up by tramway materials for road work in the devastated area, ammunition to the advance artillery positions, and carried on the work of improving the tramway roadbed and clearances so that

Mr. Perry.

locomotive-hauled trains could go forward. The lateral tracks at the rear of the divisional areas on the map (Plate XXVI) suggest the location of the forward light railway dumps; also that of the light and heavy artillery position zones. In any sector under preparation for an advance where no tram line had been located suitably to connect forward, or where conditions had changed the relative locations in this respect, the light railway troops built one or more such lines with the sole view of pushing them forward.

An intermediate area on either side of the battle line, to a depth of about ten miles, was the light railway distribution zone. This was the great dump area of corps and army, for artillery and engineer supplies. In the rear areas the light railways tapped the different supply heads of the army, with break-down connections to alternate heads. The vulnerability of all field railways was decreased by the use of many lateral tracks, which were equally useful as short-cuts.

Elsewhere on the map the German field railways formed networks of overlaid systems. That opposite Verdun suggests the extraordinary offensive-defensive ground organization at the tip of the salient, here in contact with the Hindenburg lines of field fortifications in the region of French Lorraine, which they enveloped. These may be traced roughly through the light rail-heads in the general direction northwest from Thiaucourt to Stenay, or by the battle line at the base of the St. Mihiel salient.

The German left on the Verdun salient, or the French left on the St. Mihiel salient, as the front may be viewed, occupied difficult ground for operations on a large scale in either direction. Northwest of Combres the Germans were in contact with the permanent outer defences of the French frontier—a line of forts connecting the fortresses of Verdun and Toul, here occupying the high ground east of the Meuse River and commanding the plain of the Woevre. At Combres and at Apremont the battle line entered the heights, cutting the Meuse Valley communications of the fortress of Verdun. The railway line —Commercy-Apremont-Woinville-Heudicourt-Vigneulles-St. Maurice-Combres—before the war a meter-gauge railway through Eix and Ornes to Montmedy, meandered the plain at the foot of the heights, which rise more than 300 ft. above. At Vigneulles there is a hole or valley through which a small stream, the Ruisseau de Creue, flows from the plain to the river. The Germans built a standard-gauge branch from Chambley, on the main line, crossing the Woevre to a point between Vigneulles and Creue at the entrance to the hole, which was doubled by light railways that tapped both the main line and the other standard-gauge railways out of Metz. At the end of the branch, close under the heights, they built a large depot and light rail-head; their light railway yard was located at Billy-sur le Cotes, nearby, and there were ammunition dumps, hospitals, and cantonments well apart

in the same area. The artillery in the heights mostly faced the French in the Valley of the Meuse. On the reverse slopes—toward the plain—were improvised billets. The field railways in the heights comprised two systems, one north and the other south of the hole. The system north was reached by funicular lines, from the Billy Yard and from the depot, near Creue, and by one grade line in the shape of a horseshoe curve, toward the St. Remy Woods. The system south was reached by two lines, one a grade line that climbed the southerly slopes of the Creue Valley, toward Chaillon, and the other a line that occupied the reverse slope at Heudicourt where the grade was about 20 per cent. Mr. Perry

The forward areas sloped irregularly toward the Meuse and could have been reached directly only by way of the Creue Valley. The French had good observation in that valley; a few gas shells judiciously placed in the daily shelling programme could have compelled a relocation of the supply approaches, had the valley been much used. These considerations will illustrate the following point: The location of railways in the combat area is a tactical problem and then a technical one, for which reason plans for railway lines came under the supervision of Staff agencies whose business it was to know all about the enemy and our own troops, situations, intentions, and the like, including the supply arrangements. There was not the situation of one service doing one thing, and another something else, each according to its own ideas or traditions; but all were brought together and under the same orders through a staff that possessed enormous facilities for keeping in touch with the needs at every point and at every hour and the authority to direct, in the name of the command, what should be done. For the first time in the history of the American Army, every activity of the American forces in France came under General Staff control.

Toward the Toul front were elements of a line of defense that paralleled the rallying line before referred to, from the Moselle River to Thiaucourt, running thence in a southwesterly direction to the foot of the heights, and may be roughly traced through the light rail-heads in the general direction of Woinville, which were in front of this position. The group of German railways east of Thiaucourt were in broken ground above the Moselle.

The French rail-heads appear on the map as farther back of the battle line than the German heads. The German defences were organized heavily on the first line, the French some miles back. At Flirey, the French outposts were as close as 50 yd. to the enemy trenches, but the battle line was in rear of Bernecourt, 4 miles back. Moreover, many of the French light railways served the permanent forts around Toul and Verdun. The map (Plate XXVI) is not an average section of the front. It shows the most vulnerable section of the German front,

Mr. Perry.

which will explain the great mileage of German railways there; but the French Third Army alone had 500 km. of light railway lines in its area. The American forces built about 200 km. The Germans built their light railways of materials from Belgian meter-gauge railways which they had dismantled for the purpose; they built their tramways of materials fabricated for the purpose.

The Toul front, St. Mihiel to Nomeny, was the front of main attack on the St. Mihiel salient. The standard-gauge railway, Toul-Pagny-Sorcy, was and is the main line of railway between Paris and Nancy. The lines converging to the rear meet at Neufchateau, the American main supply head of standard-gauge railway. Sorcy was an important forward rail-head; in so far as operations planned were carried out, it was most important. Sorcy, however, was not the only rail-head; there were others, both in rear of the Paris-Nancy line and forward of that line, as far forward as the battle position in rear of Bernecourt.

A rail-head is essentially a main track, a side track, a platform doubled by the side track and a good road circuit, and a small amount of storage space. For troops, supplies (food), ammunition, engineer materials, etc., the rail-heads were at different points, usually, and provided different facilities. Were a rail-head to provide the facilities necessary for handling all kinds of army traffic, it could not be worked for all at the same time, nor for any two of them regularly, though it was sometimes necessary to receive occasional shipments of different kinds at the same rail-head, not a busy one. Supply heads were generally farther forward than the troop heads and as near as possible to the center of the troop area in any case, the troop rail-heads often being on the strategical railway or *voie-de-rocade* nearest the front. Divisional artillery could be handled at a troop rail-head that could handle the horsed transport. Tanks could be handled wherever a good ramp could be had, and the Tank Corps aimed to increase the scope of its activities by providing portable ramps for the tanks. Railway artillery required sidings or spurs, not classed as rail-heads.

The Sorcy light rail-head was a supply head, disposing of two side tracks, platforms, light railway tracks and a road circuit, and had a storehouse. It was located so as to serve the Toul front and the German tracks in the Woevre, but not the Meuse-Argonne front nor the front between Metz and the Vosges.

The connecting link between the supply or other rail-heads and the trenches was a multiple one, and of different kinds of communications; but the trenches belong to the stationary phases of the war. In war of movement there were no light railways after the trench supply area had been over-run. The Germans, in Picardy, took over all the light railway area shown on the map (Fig. 1) and, in addition, ground to a depth of about 20 miles beyond the rearmost control of that system in

which there were no light railways when they took it over. Based on a situation of movement, road transport throughout the war was and will always be the connecting link between the supply heads and the troops. When a division is in the line, its supply transport waits in the rear, if there is nothing else for it to do, to rejoin the division and replenish it in an advance or during moves and rest periods. The divisional supply trains can make 100 miles per day on a circuit, if necessary. They require good roads, however, and enormous maintenance to keep them good. Mr. Perry.

The minimum rail requirements of the front were, one standard-gauge line forward in each corps area, doubled by one light railway line terminating in a broad loop in the rear of the divisional areas, and one good tram line forward in each brigade area. The minimum road requirements were, one good two-day circuit (for one-way traffic) in each corps area and in each divisional area, together with good side roads for the horsed transport and light automobiles, pack tracks for the pack mules, and paths for the carrying parties. This much was needed to work all the available transport in the normal way. In the same areas the required lines paralleled each other, with frequent spurs, as from rail to road dumps and from road to rail dumps, and cross-connections, between lines of adjacent sectors and those of the same kind in the same sector, when there were more than one (the Germans aimed at two light railway lines of supply per divisional sector). In the back areas from two to four railways for strategical moves were the main strength of the different fronts.

The rail and road arrangements, however, were practically imposed by the shape of the front in its relation to the shape of the existing systems of railways and roads; and the same is true of the canals. Much work was done after the line had stabilized to bring these systems into the service of the front by constructing combat railways and roads in sectors and also in the back areas, so that railway trains, for instance, could be brought by roundabout lines to places which would otherwise be cut off. The British front was the most difficult to get around by railway.

Light railways supplemented the motor transport columns of the rear and the motor transport trains of divisions, as well as a large but uncertain amount of motor transportation of corps and army which otherwise would have been necessary. The standard-gauge railways were brought up wherever possible so as to be within reach by the divisional trains, but they were not always available when needed most. The same may be said of good roads. Such communications as railways and roads facilitated an enemy advance as well as area operations. On the Meuse-Argonne front there were three American corps in line, of which the center corps had no road at all which was

Mr. Perry.

useful and the right corps had only one road, a muddy one. The left corps had a good road, along the easterly edge of the Argonne Forest, and this road, after repairs, had to take the army traffic which was brought to it obliquely from the center.

It is to be noted that a connection with the Chambley-Vigneulles Branch of standard-gauge railway might have been made in the Woevre by pushing a standard-gauge line forward along the right of way at the foot of the heights. Such a forward line was built as far as Woinville, at the time of the Armistice—branching from the Paris-Nancy line at Commercy. Also, to that time, the light railways out of the Sorcy rail-head could not be worked much farther than Vigneulles, within the old salient, serving the troops in the line. Sorcy, 10 miles behind the old front, was 20 miles behind the new one, between Nomeny and the tip of the salient at Verdun; but Vigneulles was too near to be used as a forward standard-gauge rail-head.

Mr. Hadden.

GAVIN HADDEN,* ASSOC. M. AM. SOC. C. E. (by letter).—It may not be out of place in a discussion of this interesting and valuable paper, to mention some of the other types of transportation which were developed by the British Army for use in the extreme forward areas in conjunction with the light railways.

Mr. Pugh has divided the light railways as operated by the American Expeditionary Forces into three zones, the third of which he terms "Tramlines at the Extreme Front, Using Animal or Man Power". This zone in the British transportation system had become of such importance during the period of fixed trench warfare that early in 1918 a special department was organized, separate from the Department of Light Railways, and given the name of "Foreways." This department, under the command of Col. Battye of the Royal Engineers, had a school and experiment station in full operation at the time of the beginning of the German drive in March, 1918. Some of the methods of transportation which were being developed at this time were extremely interesting, and might well be applicable to civil uses. In March, 1918, the writer made a brief report on some of these types of transportation to Maj.-Gen. W. C. Langfitt, U. S. A., who at that time held the office of Manager of Light Railways, as well as Commander of the American engineers with the British Expeditionary Forces. The writer lost all his personal records in France, and must rely, therefore, on his memory for descriptions of the various forms of transportation equipment. Complete details, however, may be found in the records of the War Department.

Tramways.—The tramways consisted simply of a light form of light railway, and have already been described briefly by Mr. Perry.

* New York City.

Mr Hadden.

Mono-Rail Car and Track.—The mono-rail form of transportation had been completely designed and was about to be put into experimental operation when the rapid advance of the enemy caused all further development to cease. The design called for a single 9-lb. rail, without specials, to be laid on short pieces of tie or any other material which might be available. Its principal advantages were that it required very little grading and no special work, and that it was easily transported, quickly laid, and quickly repaired. It was claimed that short breaks in the line could be crossed without any rail.

The car for this track was of special design, having two double flanged wheels in tandem, with the body, about 7 or 8 ft. long and 3 or 4 ft. wide, set very low between them. A 6-ft. horizontal pole, about 4 ft. off the ground and set at right angles to the track, at the head end of this car, in adjustable holders to allow it to be shifted from one side of the track to the other, was designed for balancing. With this leverage and the low center of gravity, one man could easily balance a heavy load. Either man or animal power was to be used for hauling.

Hamilton Ropeway.—The Hamilton ropeway was the invention of a lieutenant in the British Army from whom it received its name. It had been used for some time with great success before the writer saw a section in operation at a forward battery position near Maroc. The equipment consisted briefly of two parallel wire cables $\frac{1}{2}$ or $\frac{5}{8}$ in. in diameter, supported on upright standards. From these cables were suspended trains of carriages loaded with ammunition, etc., each on two grooved wheels in tandem and drawn usually by man power. The standard consisted of a piece of 3-in. pipe, about 7 ft. long, bolted at the lower end to a steel angle base, and at the upper end to a steel angle cross-arm. At each extremity of the cross-arm a cableway was fastened, one cable being used for operation in each direction.

The ropeway was erected in independent sections about 200 ft. in length, so that if the line was damaged by shell fire only one section would be put out of commission at a time and could be readily repaired. The cables were anchored to "dead men" at the end of each section, the terminal cross-arms being designed so that the cables from both directions passed over them in the same straight line and were fastened with special clips to form a continuous track. The carriages were coupled together in trains of a length equal to the average span between standards, so that the grade resistance caused by the sag in the cables was approximately balanced. All parts of the cableway could be easily handled and assembled under almost any conditions. The fastenings were designed so simply that they could be set up at great speed at night. The cables were drawn taut with block and tackle.

Another great advantage of this type of transportation lay in the fact that it formed an invisible target for direct fire and could only

Mr. Hadden

be traced on aeroplane photographs after a path had been worn by the men hauling the loads. When the ropeway had been in operation long enough to be plainly visible in this way, it could be shifted a short distance to new ground.

Leaming Ropeway.—The Leaming ropeway was similar to the Hamilton ropeway in design, with the principal differences that a power cable was used for hauling instead of man power and that the standards were X-shaped and built of wood. The advantage of this ropeway over the Hamilton ropeway was principally the fact that it saved man power, and also saved casualties where exposed positions were crossed. It was, however, not nearly so satisfactory as it was subject to frequent break-downs and the whole system would be thrown out of operation by a break at any one point. It was, moreover, much more cumbersome and difficult of erection, and motive power was not always available.

The principal aim of all these systems of transportation which were being developed by the British was the saving of man power. The shortage of men was so acute that every effort was made to furnish mechanical aid to muscular effort. One of the simplest and yet most effective aids of this nature was an ordinary wheelbarrow, fitted with a small pilot wheel forward of the main wheel and designed to clear the ground about ¾ in., in normal position. This contrivance was effective for hauling loads over uneven ground, and particularly on duckboards in the trenches. With a heavy load, the ordinary wheelbarrow would sink into a small shell hole or drop between the duckboards, where one or two were missing or broken, and it would be extremely difficult for one man to haul it out. With the addition of the small pilot wheel, however, a load could readily be raised at the farther edge of the hole in much the same way that a baby carriage is raised from pavement to curb.

Mr. Jonah.

FRANK G. JONAH,* M. AM. SOC. C. E. (by letter).—The writer is greatly pleased with the interesting discussions which his paper has elicited, and expresses his thanks to those who have contributed so much of value to the subject. Taking the discussions in order, it is desired to refer in turn to points brought out by the contributors.

It is true, as stated by Mr. Hay, that the French had developed the light railway prior to the World War, but for use in and around permanent fortifications. There were some short lines connecting Toul with outlying forts, also some at Epinal, Verdun, and doubtless elsewhere. For instance, at Toul there was a line leading from the arsenals up to the fort on the summit of Mont St. Michel, a hill 300 ft. high, immediately north of the city, on which the French maximum grade of 10% was used. The writer, with a party of officers, made the ascent to Mont St. Michel in December, 1917, in a wagon or gondola

* St. Louis, Mo.

car, hauled by a Pechot locomotive. However, for tactical use, light railways had not been considered until the fact was demonstrated by actual experience that the front line could not be adequately supplied by motor transport. As to the standard section of rail, the writer knows that various weights were used, both by the French and Germans, namely, 5, 7, and 9 km. per meter. Mr. Jonah.

The studies of results by American engineers led them to believe that bolted track was better than riveted track where steel ties were used, because when a section of riveted track was damaged by wreck or shell fire, the whole section had to be taken out; but with bolted track, the bent rail or bent tie could be readily removed and repairs quickly made. In December, 1917, the writer visited the French light railway shops at Chartres, and noticed a large reclamation yard, where German prisoners were cutting apart bent up and twisted sections of riveted track which had been salvaged from the front. He also noticed German prisoners assembling new bolted track, of rails weighing 20 lb. per yd., with 8 steel ties, 1 m. long, to 5-m. section of track, which material was of American manufacture. This track was of 60-cm. gauge, and was being sent to the Italian front. The prisoners were given task work, and 4 men were required to bolt up 27 sections in 9 hours; thus, 48 men assembled a little more than 1 mile of track per day.

Col. Green has treated of the necessity of light railways in forward areas, and speaks from experience, as he had charge of the operation of a system of these lines on the British front in Picardy, in the autumn of 1917, and worked so close to the front lines that he used both steam locomotives and gasoline tractors and had much experience in repairing lines damaged by shell fire.

Gen. Black has called attention to the lack of preparedness when the United States entered the war, and pays a deserved tribute to the engineers from civil life who so promptly responded to the call, and who, working under the small nucleus of the Corps of Engineers of the Regular Army, were able to evolve such a large and efficient body as the Corps of Engineers, A. E. F., at the close of the war. The writer believes in universal military training and knows, too, that the efficiency of the men in the Corps of Engineers is the result of their years of experience on the great public works of the country.

It was the writer's great privilege to serve under five very distinguished members of the Corps of Engineers of the Regular Army. They were all good engineers, had handled large works in the United States, the Philippines, Panama, and Cuba, and knew how a job should be handled and how men should be handled, and the writer believes with Gen. Black that the status of the Engineer Corps should not be changed with respect to its employment on public works. He believes

Mr. Jonah. that its sphere of employment in peace times should be enlarged, and that a young graduate from West Point could profitably spend a year, for instance, in the transportation department of a large railway system.

It is a pleasure to pay tribute to the splendid work of Col. Wilgus. His Commission, after a study of only a few weeks, cabled a requisition to the United States, early in July, 1917, for light railway track material and equipment, and it is only the simple truth to state that it was adequate in quantity and suitable in quality for all the needs of the American Army up to the signing of the Armistice. In this connection, great credit is due also to Gen. Felton's organization in Washington for its prompt action when the requisition reached it. This organization hurried out the designs, placed orders, secured the material, and had it shipped with no delay whatever.

It is, of course, a fact that much additional equipment and track material was ordered and some received in France prior to the cessation of hostilities. This was done, however, on the assumption that the war might last through 1919, and it enabled the United States Government to supply the French with considerable quantities of material, and a great quantity was piled up in American storage yards when the war ended.

The writer's experience with this material enables him to state that it was entirely satisfactory when properly used. Perhaps a gauge of 30 in. would have been better, as giving greater stability to track, but that was out of the question with a 60-cm. gauge in use by all the other armies. Several of the discussors have referred to the frequent derailment of the locomotives. This was due, usually, to inexperienced men trying to run them too fast on poor track. One lesson is fundamental, track must be ballasted and kept in good surface; another is that trains should not be run faster than 8 miles per hour.

As to the use of the light railway after position warfare ceased and open warfare was resumed with the retreat of the Germans in the closing months of 1918, the writer contends that the light railways did not cease to function by any means, and were as necessary as in the previous months. It will be seen by reference to the map (Plate XXVI) that the Germans had a very great mileage of light railways which they were unable to destroy in their retreat. It was only the work of a few days in some cases, and a few weeks in other places, to connect the American system with that of the enemy and haul supplies to the troops in the occupied area formerly held by the Germans. Thus, supplies were hauled from the Sorcy rail-head to Conflans, about 60 km. over light railways, and from Bernecourt and other rail-heads long distances into the regions formerly in the hands of the enemy. To be sure, this was not an ideal way of hauling supplies. Light railways were not designed for long hauls, but in the absence of standard-gauge lines (the Germans had greatly damaged them) and of sufficient motor

transport, the light railways saved the situation; and, again, their construction would be justified, if for no other reason than their use in salvage operations in the devastated areas. Mr. Jonah.

Mr. Dunham has certainly contributed something of interest, and it will doubtless be news to most engineers to learn that 2-ft. gauge lines are used successfully for the regular transportation of passengers and freight in the United States. The lines in the State of Maine, which he has described, must be very well constructed for the speed made over them. Their grades and curves conform to the practice generally followed in France.

As to light railway artillery trains the writer saw such a train that was regularly used by the French. It was in the shop at Toul, and had just returned from the front. In it were mounted 6-in. guns on flat cars provided with drop sides, or outriggers, to steady the car when the gun was discharged. The French moved this train out at night, shelled the Germans awhile from one position and then moved to another, and claimed that it was a great success.

The writer is sure that Capt. Pumphrey will agree with him that most derailments were due to bad track and excessive speed, and he will also agree that during the time the 21st Engineers were operating the light railways from Sorcy, prior to the St. Mihiel drive, or from March 1st to September 1st, 1918, comparatively little trouble was experienced on this account. The track was well kept up during these months, and after the drive, the connecting tracks to the German lines might have been more quickly constructed had sufficient troops been available. The writer believes that the importance of these connections was not fully appreciated at General Headquarters, and, consequently, troops and material were not at hand to make them promptly.

Replying to the criticisms of Capt. Hoyt: The light railway cars were heavier than they should have been, and, doubtless, some improvement could be made in the locomotives and tractors, but such defects as they had were by no means fatal, and the difficulties experienced were the fault of the men running them, rather than the equipment itself. As to speed at which tracks could be constructed, the time given by the writer is for a road complete, including ballast; whereas, Capt. Hoyt gives some figures for track-laying on an old road-bed, with such added work as was entailed in filling shell holes, mine craters, blown-up culverts, etc., which is not comparable with the work of building a new light railway.

The location of the railway shops at Abainville served the St. Mihiel and Argonne fronts better than any other location that could have been selected to serve both, and when established was 37 km. back of the line. Location closer to the enemy's front line than 30 km. was not permitted. These shops were so nearly complete that they were

Mr. Jonah.

able to set up and keep sending a constant stream of equipment and supplies of track material to the First Army during the Argonne battle. It is true that it may have taken three days to get it up on its own rails, but that was much better than not getting it at all, and to have undertaken to load light railway track material and equipment on standard-gauge cars and send it around that way would have meant weeks, instead of days. There was a great shortage of equipment on the standard-gauge railways, and, often, it took days to get a car for loading.

The writer noticed that various systems of light railway on the French front were not connected. Each system radiated from some standard-gauge rail-head, and there were no connections except such as might be had by transferring equipment or material at the broad-gauge rail-head and moving it around to another sector on the standard-gauge track. The writer suggested the advisability of connecting the various light railway systems, which could be done by building short stretches of 60-cm. track, and it was such a connection finally made that enabled light railway equipment and track material to be taken from the St. Mihiel Sector to the Argonne.

The reports required by the Department of Light Railways were for the purpose of keeping a necessary quantity of supplies moving to the front and placing orders for a sufficient stock from the States to take care of the requirements. The Department did not expect that the American operations would be helped materially by fuel captured from the Germans, but from such reports were enabled to form a very fair idea of needs long before anything was captured from the enemy.

The writer disagrees with Capt. Hoyt that ample troops and material were provided for making quick connections across "No Man's Land", and refers to the experience of Capt. Pumphrey at Flirey. The writer made some personal observation on the ground of the progress made in changing the meter gauge to standard gauge running north from Commercy. There were not enough men on that job to put it through in anything like a reasonable time; and that, as has been pointed out in the discussion, was one of the partial failures of American engineers, they did not extend the standard-gauge lines as quickly as they should, largely by reason of a limited number of troops on railway construction.

The writer is indebted to Maj. Pugh for his very complete and able discussion of the subject. He is particularly pleased that Maj. Pugh has treated so fully of the work at the front, and has so clearly emphasized the different standards of construction that must, of necessity, prevail. The views accompanying his discussion add greatly to the value of the paper as they give a picture of conditions as they existed in the extreme forward areas.

Mr. Jonah.

The writer believes, with Maj. Pugh, that 30 in. would be the proper gauge for a light railway, and sections about 16½ ft. long with eight ties bolted to the track would be ideal for work at the extreme front; and, perhaps, to make up for extra weight in the tie, the rail section might be reduced to 20 lb. per yd. It is desirable to have sections of track that can be handled readily by a squad of men. The 5-m. section with a 25-lb. rail used by the Americans, weighs 461 lb.

The writer believes that the suggestions of Lieut. Gregg, referred to by Maj. Pugh, as to light railway equipment, are good. The French light railway truck was the most efficient piece of equipment used by any of the armies.

TYPICAL ARRANGEMENT OF LIGHT RAILWAYS FOR AN ARMY.
(French Manual, April 1st, 1918)

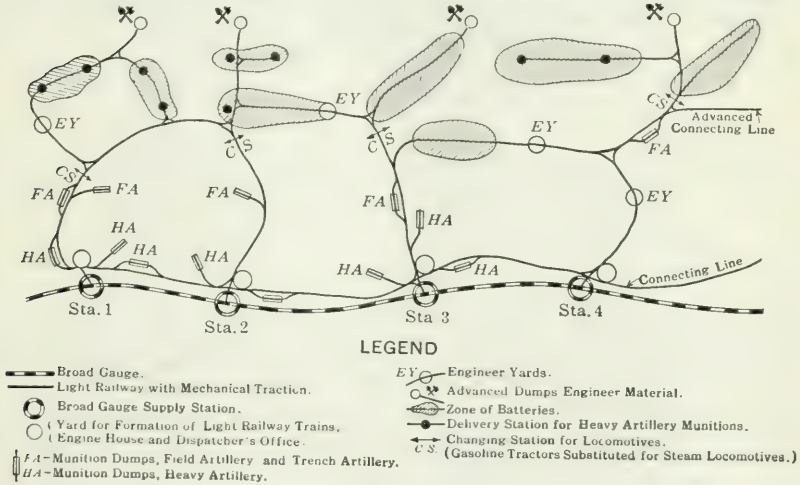


FIG. 22.

The discussion on track shows clearly the necessity of liberal quantities of ballast. In a country as muddy as Flanders, where the British had about 40 men to 1 mile on maintenance, a railway of any kind is soon impassable without ballast. A highway also requires ballast in such a country, and as it was raining a greater part of the time in France, the light railways were built and operated under the most severe natural obstacles. It is conceivable that in another theater of war these great difficulties due to almost incessant rainfall would not obtain.

Maj. Perry has stated very clearly the conditions that obtained at different times on the sectors occupied by the Americans. No doubt the ideal arrangement of a system of light railways would be as suggested by the French in their Manual of April 1st, 1918. Fig. 22

Mr. Jonah. gives a clear idea of the use to which a light railway system might be put.

Sorcy, of course, was only one rail-head and designed to serve a certain sector, and has been described as typical of what a rail-head should be.

The writer fully agrees with Maj. Perry that a standard-gauge line should have been rushed north from Commercy to Vigneulles where it would have connected with a standard-gauge line constructed by the Germans; this might have been done by converting the meter gauge to standard gauge, which work has been referred to and which was carried north only as far as Woinville.

The writer is pleased with the discussion of Mr. Hadden, as he brings out fully some types of transportation in the forward areas that were being developed and would have had an extended use in the American Army had hostilities continued.

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Paper No. 1452.

REPORT ON CAUSE AND CORRECTION OF FOUNDATION TROUBLES OF BOX FACTORY AT NATCHEZ, MISSISSIPPI.

By C. E. Smith, M. Am. Soc. C. E.

With Discussion by Messrs. Charles H. Miller, J. E. Willoughby, George L. Dillman, and C. E. Smith.

Synopsis.

During 1917 a large box factory was built on a bench of land adjoining the east bank of the Mississippi River, at Natchez, Miss., 75 ft. below the top of a bluff and 150 ft. above the river. The owner was not represented on the work by an engineer or building inspector. The work was carried out by an "engineering contractor" on a cost-plus-fee contract. Following the completion of the plant, settlement occurred in the foundations under practically all the buildings and machinery, and it appeared that the entire building site would slip down the hill.

Various causes for the settlement were suggested, such as erosion of the Mississippi River bed under water, settlement of the ground, due to fine under-soil carried out through springs by the ground-water, slipping of pervious strata above on the impervious stratum below, etc. Investigation disclosed that neither the erosion of the Mississippi River nor the action of escaping ground-water caused the trouble, which was found to be due to softening of the top 20 ft. of soil by water that could not escape through impervious blue clay under it.

The trouble was corrected by excavating trenches through the top soil to the blue clay, installing porous tile and wood box-drains and back-filling the trenches to the surface with cinders and gravel containing a large percentage of sand. Immediately following the completion of the drainage system, all ground movements stopped, the defective foundations were corrected, additional boilers, engines, and other facilities were installed, and no further foundation trouble has occurred. A total expenditure of about $20 000 for drains stabilized the soil and made it possible to continue the operation of the factory.

More than $25 000 had been spent in a futile effort to stop the movement by driving timber piles into the moving ground, and the plant was operated at very low efficiency on account of frequent shutdowns, resulting in large operating deficits each month. Immediately following the stabilization of the foundations the plant overcame all deficits and showed a profit.

During 1917 a large factory for the manufacture of wood boxes, tubs, and veneer was constructed under contract on the east bank of the Mississippi River, at Natchez, Miss. The general plans were prepared in the office of the owners at Chicago, Ill., who were represented on the construction work by a bookkeeper; the future plant manager visited the work occasionally. The owners did not have an engineer or building inspector on the work, as it was thought that the "engineering contractor" would automatically provide the engineering and inspection. Later developments indicated that special engineering knowledge and advice during the construction period were conspicuously absent, and nearly resulted in the wrecking of the plant.

Ground Conditions and Movements.

At the site of the factory the Mississippi River has a vertical rise and fall of about 50 ft. The bank slopes upward about 1 in 5 for 750 ft. back from the high-water line to the highest ground 150 ft. above. A cross-section of the bank between the river and the top of the bluff, at the site of the plant, is shown in Fig. 1.

The surface of the sloping bank consists of a thin layer of black loam, under which is yellow clay of indefinite depth. In this clay appear, at several depths, and in irregular thicknesses and extent,

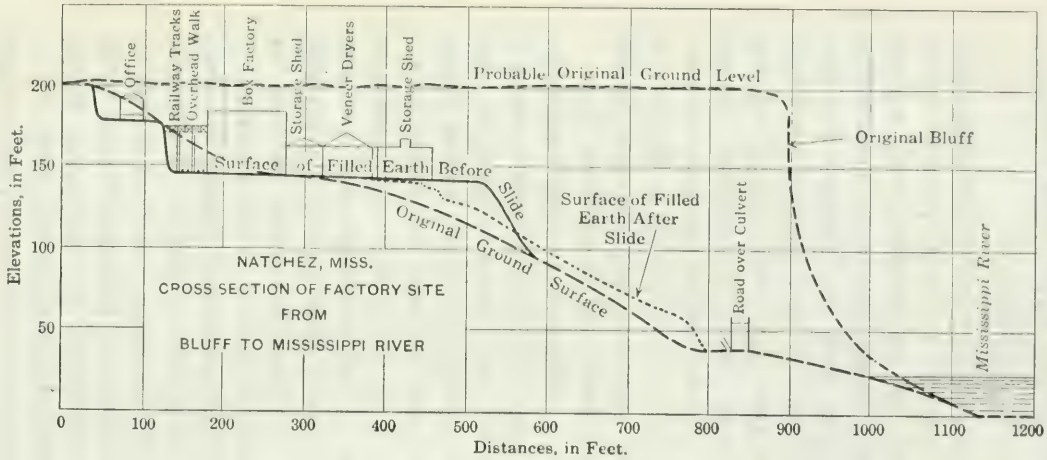

Office
Railway Tracks
Overhead Walk
Box Factory
Storage Shed
Veneer Dryers
Storage Shed
Probable Original Ground Level
Original Bluff
Surface of Filled Earth Before Slide
Slide
Original Ground Surface
Surface of Filled Earth After Slide
Road over Culvert
Mississippi River
NATCHEZ, MISS.
CROSS SECTION OF FACTORY SITE
FROM
BLUFF TO MISSISSIPPI RIVER
Elevations, in Feet.
Distances, in Feet.
200
150
100
50
0
100
200
300
400
500
600
700
800
900
1000
1100
1200

FIG. 1.

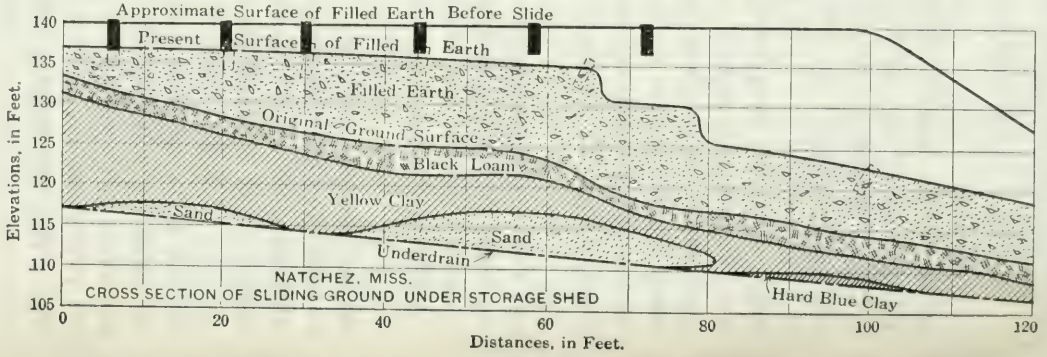

Approximate Surface of Filled Earth Before Slide
Present Surface of Filled Earth
Filled Earth
Original Ground Surface
Black Loam
Yellow Clay
Sand
Sand
Underdrain
Hard Blue Clay
NATCHEZ, MISS.
CROSS SECTION OF SLIDING GROUND UNDER STORAGE SHED
Elevations, in Feet.
Distances, in Feet.
140
135
130
125
120
115
110
105
0
20
40
60
80
100
120

FIG. 2.

various strata of coarse sharp sand, fine quicksand, mixtures of this fine sand and clay, and sloping layers of blue clay. The mixture of fine sand and clay is very firm and hard when dry, but when saturated with water it becomes a typical quicksand.

For many years, beyond the memory of the oldest settler, this typical Natchez river bank has been sliding or flowing toward the river, causing continual disturbance and subsidence of the ground surface on the slope. The movement of the ground was a miniature reproduction of glacial action. A typical cross-section at a point where the greatest movement took place is shown in Fig. 2.

Residents over 80 years old remember being told of the time when the slope was higher and level and occupied by buildings. It appears that the high ground originally extended out to a bluff on the river bank, and that the present slope is the result of the sliding to which reference has been made.

At extreme low water, or zero gauge of the Mississippi River, a porous stratum of cemented gravel and sand with a small mixture of iron ore, locally referred to as "ironstone", outcrops along the entire Natchez water-front and gives forth an abundant flow of clear cool water through numerous springs.

This emergence of the underground water 200 ft. below the high ground above has been commonly held as the reason for the sliding and settling of the sloping bank. A careful examination of the porous stratum and of the springs, together with other conditions and information, convinced the writer that the sliding and settling were not due to this cause. The water emerging from the porous strata comes out perfectly clear and does not carry any sediment. If, in fact, fine sediment were being carried out from deep levels by this underground water, undoubtedly caves would be formed, which, in the fine, easily broken-down soil that overlays the cemented gravel, would ultimately result in chimneys and sink-holes. There are no such chimneys and sink-holes at Natchez.

Bank Erosion by Mississippi River.

Another theory which was advanced to account for the sliding ground is that the erosion of the river bank below low-water level caused the bank to subside and gradually flow toward the river. This theory was strengthened by the fact that within the memory of present

residents of Natchez the river has made extensive inroads on the lower portion of the city. The present high ground on which the city is located was formerly separated from the river by an area of comparatively low and level ground, four or five city blocks in width, fully occupied by buildings, known as "Natchez-under-the-hill".

Approaching Natchez from up stream, the river makes a long sweep or bend and takes a direction nearly at right angles to the bank, then turns in another bend to follow the direction of the high ground. The low ground of "Natchez-under-the-hill", having been located in the bight of the bend, was eroded until it nearly all disappeared, barely space enough remaining for one street on the side-hill between the water and the bluff. Further erosion of the bank was prevented by mattress and revetment work placed by the United States Government. The United States Engineer Corps took numerous and frequent soundings along the front, a study of which indicates that since the revetment was placed, no further erosion has taken place. The bank down stream from the revetment, opposite the site of the factory, has not eroded to any appreciable extent, and it does not appear that the sliding and subsidence of the sloping bank above high water was due to the erosion of the banks by the river. The river is quite deep, the bed being 60 ft. or more below low water.

According to Mark Twain's reference to "Natchez-under-the-hill", its disappearance might have been a well merited fate. In his "Life on the Mississippi", about 1880, he refers to it as follows:

"Famous Natchez-under-the-hill has not changed notably in twenty years; in outward aspect—judging by the descriptions of the ancient procession of foreign tourists—it has not changed in sixty; for it is still small, straggling and shabby. It had a desperate reputation, morally—in the old keel-boating and early steamboating times—plenty of drinking, carousing, fisticuffing, and killing there, among the riff-raff of the river, in those days. But Natchez-on-top-of-the-hill is attractive, has always been attractive."

Plant Site and Buildings.

A site for the factory was prepared by leveling the slope about 60 ft. below the top to a bench approximately 350 ft. wide, the deepest cut in the bluff amounting to nearly 30 ft. and the highest fill about 25 ft.

The factory buildings, covering about 2 acres, and including the boiler-house with brick stack, engine-house, veneer mill, kiln veneer dryers, storage sheds, box and tub factories, soaking vats, elevated water-tanks, and appurtenant log conveyors and tramways, derricks, and saw-mills, were built on the graded ground. The construction was completed, and operation of the plant commenced early in 1918.

Failure and Movement of Foundations.

Less than a year after the completion of the plant, and during the rainy season, the ground sheared vertically along several lines normal to the slope of the hill-side, with a settlement of from 2 to 15 ft., and slid down the hill-side, carrying large sections of surface earth with growing trees as far as 200 ft.

A typical crack in the ground preceding a slide is shown in Fig. 4. The lower slope of the sliding ground is shown in Fig. 6.

Shortly after operations commenced, the foundation walls under the boiler-house broke, and the river end settled about 1 ft. and moved toward the river 1 ft.; the brick walls fell down, the brick boiler settings of two 300-h. p. boilers were shattered, and the boilers pulled away from the breeching. The 90-ft. brick stack settled and caved slightly, and the water-tank footings showed a slight settlement.

The plant was originally operated by a 550-h. p., Corliss engine and line shaft. The engine foundation settled and threw the engine out of setting. The concrete foundations under the 6-in. jack-shaft settled several feet and slid 5 to 10 ft. entirely outside of the engine-house. The engine-house foundations on the river side settled and slid completely out from under the engine-house. The movement of these footings is indicated in Figs. 7 and 8.

Practically every footing under the storage shed settled and moved out of place. The outside row on the river side settled over 15 ft. and slid out nearly 30 ft. About seventy footings settled and moved. Figs. 3 and 5 show the nature and extent of this settlement and movement.

The settling ground extended back to the foundations of the heavy veneer mill and planers and the kiln veneer dryers; it broke sewer and water lines, caused frequent long shut-downs, made it impossible to get on a production basis, and, for a while, it appeared as if the

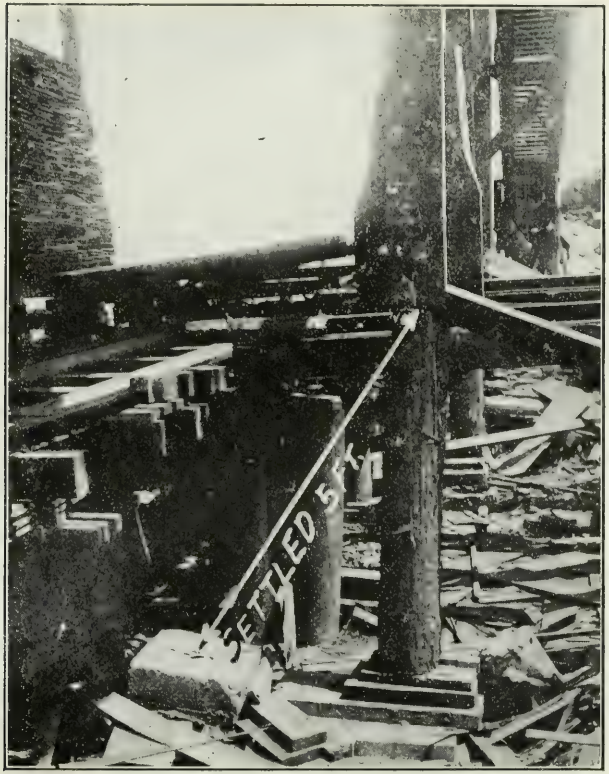


FIG. 3.—CONCRETE FOOTINGS UNDER WEST EDGE OF STORAGE SHED.

FIG. 4.—TYPICAL CRACK IN GROUND PRECEDING SLIDE.

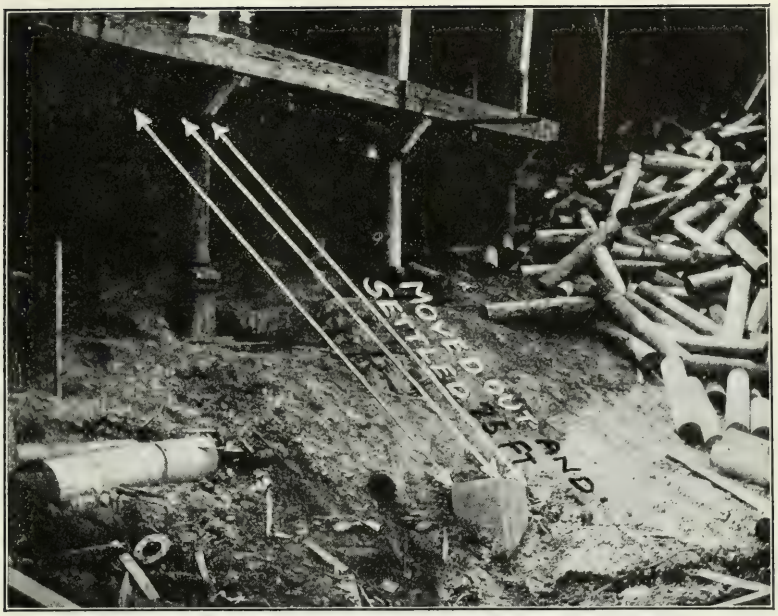
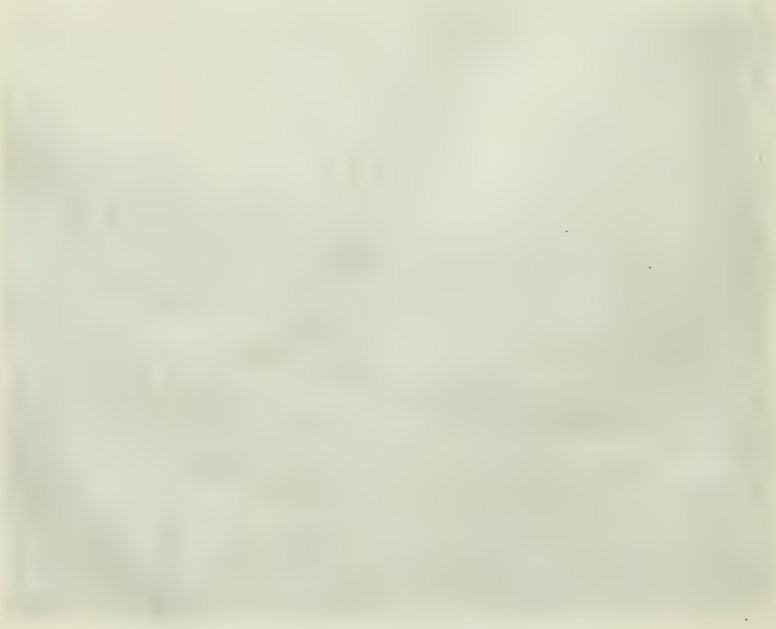


FIG. 5.—CONCRETE FOOTINGS UNDER WEST END OF STORAGE SHED.

FIG. 6.—LOWER SLOPE OF SLIDING GROUND.



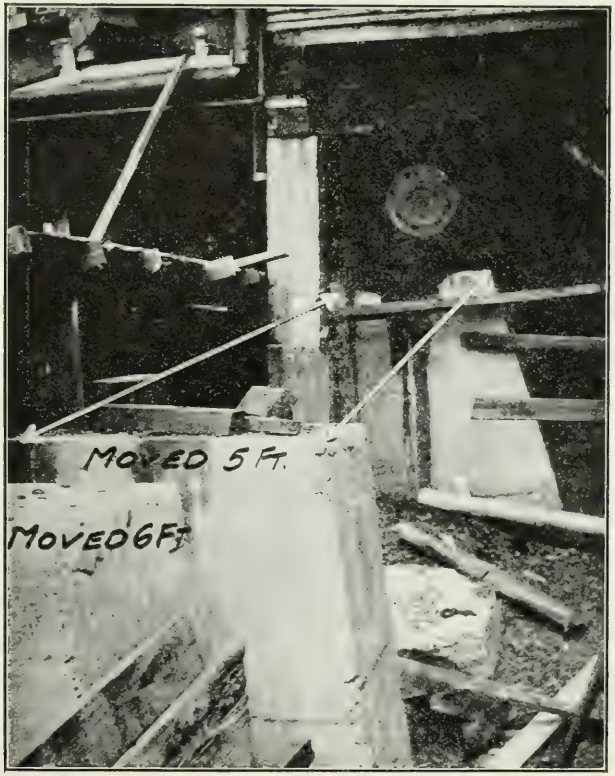


FIG. 7.—CONCRETE FOOTINGS UNDER MAIN LINE SHAFT.

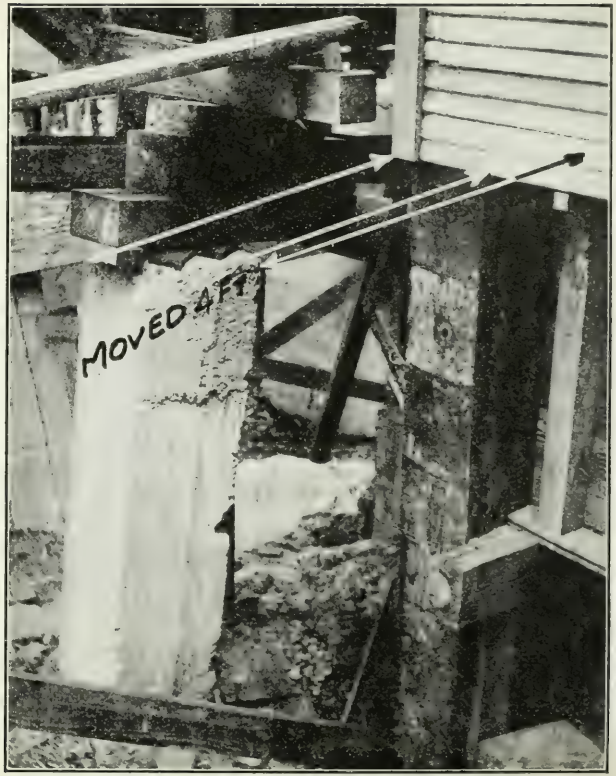


FIG. 8.—CONCRETE FOOTING UNDER CORNER OF ENGINE HOUSE.

enterprise would have to be abandoned, and the plant moved to another site.

Emergency Repairs.

Ineffectual methods were adopted to stop the sliding ground. Timber piles were driven into the ground, but they were carried right along, tipped over, upheaved and literally thrown out of the ground by the movement, as shown in Figs. 9 and 10.

After closing down for sixty days, the factory was continued in intermittent operation with the greatest difficulty by keeping gangs of workmen all over the plant jacking and propping the buildings on the moving ground. A new connection was made between the unaffected portion of the jack-shaft and the re-set Corliss engine. A new engine, with dynamo, was installed and motor drive put on several machines.

Investigation of Trouble.

After this painful experience had continued for about a year, the situation was brought to the writer's attention for the first time early in 1919, when the owner requested that an examination be made, the trouble diagnosed, the remedy prescribed and estimated, and recommendations submitted for further procedure.

After a careful examination, the writer concluded that neither the erosion of the river nor the springs at low-water level had anything to do with the settlement, but that it was due to local conditions, and that the ground movements and slides could be stopped by a system of under-drains. The filled earth was a very loose mixture and contained a quantity of rubbish which prevented it from packing and forming a hard surface such as would permit water to flow off rapidly. On the contrary, the surface was broken by cracks and depressions which collected the water and caused the filled earth to become rapidly saturated to the depth of the original ground surface.

On this less pervious surface the water collected, saturated the loose loam and clay to a slimy mass, 1 or 2 ft. thick, and started to flow down the slope. Farther down the hill-side at the lower end of the fill, the water came to the surface washing with it a mass of soft mud. As this mud oozed out, the earth above settled and this, in turn, became saturated and started to flow. Where the soft earth at the level of the original ground surface did not flow enough to cause a vertical settlement, it formed a slippery layer on which the loose earth above could slide.

Conditions which were bad enough after weeks of torrential rains common in the lower Mississippi Valley, were made worse by surface drains both for storm-water and for water resulting from the operation of the plant and, after the first slight movement of earth, by a number of leaks in underground water pipes and sewers.

The slide as well as the vertical settlement was further augmented by hundreds of tons of log cores from the mill, which were piled on the hill-sides near the top of the slope. Besides placing a great weight on top of the moving earth, these cores furnished crevices and openings in the ground surface through which the water soaked into the earth instead of running off.

Construction of Under-Drains.

A comprehensive system of porous under-drains, with a combined length of 2 870 ft., was placed under and around the principal buildings of the plant. These drains consisted of tile pipes or perforated wood boxes placed at depths of from 10 to 25 ft. in trenches, which were then back-filled with gravel or cinders. The trenches averaged 30 in. in width, which was as narrow as they could be dug conveniently to the required depth.

Tile pipe of sizes from 6 to 12 in., which could be placed more cheaply than wood box, was used in all lines where the earth was not moving; but at points where movement was taking place it was found that sections of pipe pulled apart too easily, and continuous wood box was used instead. Wherever possible this was kept below the plane of the slide.

This wood box was built of 2 by 12-in. cypress planks, the sides resting on the bottoms so as to make the interior dimensions 8 in. wide by 12 in. high. The sides and top were perforated with 1-in. holes in two rows, at 8-in. intervals in each row. The joints between the side and bottom planks of the box were well caulked with oakum and then covered with narrow lath. These joints were broken so as not to have successive planks in the side to join at the same section at which planks joined in the top and bottom; all were held by 2 by 12-in. scabs, 3 or 4 ft. long, well spiked at both ends so as to keep the box from pulling apart during slight movements of the earth. The lumber used was a low grade of pecky cypress which was found to be quite satisfactory.

FIG. 9.—TIMBER PILES DRIVEN IN EFFORT TO STOP EARTH MOVEMENT.

FIG. 10.—TIMBER PILES DRIVEN IN EFFORT TO STOP EARTH MOVEMENT.

From the lower ends of the under-drains the flow was carried down the hill-side in open flumes of wood, of the same cross-section as the box underground. Points at which the drains came to the surface were selected, on steep slopes, so as to have the drain-pipe or box pass through as short lengths as possible of the surface earth, as this was constantly in motion and tended to pull the sections of pipe and box apart. The open flumes were placed on timber bents spaced 10 or 12 ft. apart. The flume was kept from disjointing at each slight displacement of the bents due to movement of the earth. The bents could be put back in place after each movement without injuring the flume.

After the completion of the under-drains the earth near them, especially on the hill-sides, was graded to slope uniformly toward the gravel-filled trenches through which the water passed to the under-drains.

The flow through the several drains was measured daily during their construction, and that through each drain was found to decrease gradually for two or three weeks after its completion, which indicated that the level of the ground-water was being slowly lowered. The total underground seepage after the drains had been in place sufficiently long to have reached their minimum of flow, and after defects in the surface drainage had been corrected, amounted in the dryest weather to about 12 000 gal. per day from an area of about 4 acres, 2 acres of which are covered by buildings the drainage from which is otherwise disposed of. This was increased to about 100 000 gal. per day in very wet weather.

Movement of the ground stopped promptly at one point after another as the drains at these respective points were completed, until the work was finished. After completion, not the least movement could be noticed at any time. The earth, which formerly was muddy and soggy from one rain until the next, now dries rapidly after each successive downpour and is perfectly stable. The plant, valued at $800 000, in serious danger of ruin, has been made safe for operation by an expenditure of less than $20 000.

Although several months have elapsed since the completion of the drains, there has been no further trouble. The drains were placed

during the early summer of 1919. In February, 1920, the manager of the factory advised as follows:

"I am very pleased to report that even in the face of the continuous rains all through the summer and up to the present time, even in a heavy rain which we had in January, when it rained about 7 in. in two days, there was absolutely no movement in any part of the hill where the drains were put in."

Cost of Drainage System.

The cost of the work was as follows:

Excavation per Cubic Yard:	
Handling earth (in addition to actual excavation, this includes an average haul of 250 ft. for all excavated material)	$2.00
Miscellaneous items (this includes bracing the trench, foreman's supervision, trench-jacks, lumber, and rubber boots)	1.35
Total	$3.35
Back-filling per Cubic Yard:	
Labor (this includes handling of gravel from railway cars to the several drains, with an average haul of 250 ft.)	$1.90
Gravel, including freight	2.10
Total	$4.00
Placing Wood Box per Linear Foot:	
Labor (time of carpenters and laborers building the box, and placing it in the trench)	$0.65
Cost of timber	0.20
Total	$0.85
Placing Tile Pipe per Linear Foot:	
Labor (time of foreman and laborers carrying the pipe from stock piles to the drains, and placing it in position)	$0.20
Tile pipes, including freight	0.35
Total	$0.55
Cost per Linear Foot on All Operations	$7.20

The trenches reached a maximum depth of 25 ft., with an average depth of 15 ft. All trenches were braced. In many places the soil ran through the cracks like soft ice cream.

The unit costs for material and labor were as follows:

Cost of Labor per Hour:

Foreman	$0.75
Carpenter	$0.50
Negro laborers (10-hour day)	$0.25 to $0.30

Cost of Material per Cubic Yard:

Gravel ($1.20, freight $0.90)	$2.10
Cinders (loading and switching charges only)	$0.80
Lumber, pecky cypress, per 1 000 ft. b. m.	$30.00

Tile Pipe:

Size, in inches.	Pipe.	Freight.	Total.
6	$0.19	$0.05	$0.24
8	0.27	0.07	0.34
10	0.37	0.09	0.46
12	0.58	0.12	0.70

Other Plant Improvements.

When work on the under-drains had progressed far enough to convince the owners of success, work was started on the reconstruction of the damaged portions of the plant and on several additions. The old foundation of the drag-saw used to cut the rough logs to convenient lengths for the lathes, had to be entirely replaced and the engine operating the saw re-set. The foundation for this engine and saw was 20 by 28 ft., and consisted of twenty-four timber piles with an average penetration of 25 ft., capped with a 24-in. slab of concrete.

A small saw-mill was built adjacent to the drag-saw. The point of the hill on which this was located had not yet been cared for by under-drains and was still moving at the time the mill was built. For this reason, it was decided to place for a foundation a heavy frame of 12 by 12-in. timbers which could be jacked up and leveled after slight movements of the earth until such time as it would be quite stable, when a more stationary foundation could be placed.

In order to join this saw-mill with the veneer mill and the drag-saw with the boiling vats, conveyors and a heavy timber platform 9 ft. high were built to replace an old conveyor and platform which had been completely wrecked by the slide. All these structures were placed on timber piles, with penetration well into the original solid ground beneath the slide. The piles under each conveyor were placed at 8-ft. centers in each of two rows 5 ft. apart. Under the platform they were at 6-ft. centers in both directions.

All water pipes and sewers were seriously damaged by the earth movement and required extensive repairs. The fire protection sprinkling system was entirely removed from underground and put overhead.

The additions to the plant consisted of two new boiling vats, three new boilers with a combined capacity of 1 100 h. p., a new self-supporting steel stack, 8 ft. in diameter and 135 ft. high, and a new engine and 500-kw. generator. The boiling vats were each 9 ft. by 40 ft. over all, with a depth inside of 10 ft. The walls tapered from 9 in. thick at the top to 18 in. thick at the bottom, and each rested on a single row of timber piles driven at 3-ft. centers. The piles were capped with a 2-ft. floor slab.

The boilers, the stack, and the new engine and generator were not placed on piles, as the earth at these points was considered to be stable enough after the completion of under-drains in this vicinity to use the customary spread foundations of concrete.

Supervision of the Work.

The trouble was investigated and the remedy prescribed by the writer; the construction work of putting in the drains and the foundation work for correcting the defects and for the additional mechanical and electrical equipment was supervised by Assistant Engineers M. H. Doyne, Jun. Am. Soc. C. E., and Mr. G. C. Hetlage.

No contract was let. The materials were purchased by the Box Company, the labor was carried directly on its pay-roll, and worked under the Assistant Engineers. The additional mechanical and electrical equipment was specified by D. MacKenzie, Consulting Engineer, of Chicago, Ill., and installed by Mr. Robert Warmack under the supervision of Mr. MacKenzie.

The investigation, installation of the drainage system, correction of defects, and additions to the plant were carried out under the

general supervision of J. W. Drissen, General Manager of the Box Company.

Conclusion.

The employment of competent engineering assistance during the construction period would have avoided all the later troubles and expense. The employment of competent engineering service immediately after trouble first developed in the foundations would have saved large sums expended in experimenting and would have avoided the greater portion of the operating deficits. The plant could have been put on a paying basis much sooner with a great saving to the owners.

DISCUSSION

Mr. Miller. CHARLES H. MILLER,* M. AM. SOC. C. E. (by letter).—The conditions described by the author are very familiar to the writer and recall the trouble experienced ten years ago or more in this immediate vicinity in an effort to keep the track of the Natchez and Southern Railway Company in reasonably fair alignment without being required repeatedly to throw it back up the hill.

The writer is pleased to note that Mr. Smith reached the same conclusion that was arrived at several years before with respect to the facts that neither the underground water seeping out near low-water level in the Mississippi River, nor the erosion of the river bank below low-water level were the prime causes of the trouble, but that it was due to rain-water which fell on the immediate surface, causing the loose earth to become soft and to slide on smooth layers of clay located at different distances below the surface of the ground.

The treatment applied in this case differed from that described by the author in that a system of open wooden box-drains was placed on the surface of the ground, which was kept smoothed down and sloped toward these wooden drains. This resulted in carrying the rainfall off very quickly and, therefore, preventing it from soaking into the soil; it stopped the sliding, was comparatively inexpensive in first cost, and was maintained cheaply through the efforts of the section gang.

It should be noted, however, that the sliding occurred where the surface area from which underground water could originate was comparatively small, and that there was no big surface farther up the hill with which to reckon. This, the writer understands, was not the case with the box factory site, where, if surface water was to be handled, it would have meant a much greater problem.

It should be stated, also, that the railway line was located along the edge of the bank and, therefore, near the river which at high stages caused wash or erosion at the foot of the bank (the erosion being caused principally by wave wash from high winds) and that, on account of this erosion, it was necessary to construct a brush retaining wall to hold the toe of this bank against the attack of the river.

Mr. Willoughby. J. E. WILLOUGHBY,† M. AM. SOC. C. E. (by letter).—The failure of the piles to cure the slide at the box factory at Natchez, Miss., is in accord with the writer's experience. Piles driven into the moving mass can accomplish the cure of slides only by fastening the upper moving material to a lower stable stratum. If the upper material is of any considerable mass, piles are necessarily ineffective. The anchorage

* Little Rock, Ark.

† Wilmington, N. C.

in the stable stratum is insecure and the piles cannot resist any considerable force applied normal to their length. Mr. Willoughby.

The author has been happy in looking for the cause of the slide and in applying the true engineering remedy, bearing in mind the materials available at Natchez. The writer has accomplished good results in the cure of sliding embankments which support railway tracks by digging open trenches deep into the saturated stratum of the embankment, and filling the ditches with stone of a size that can be handled readily by two men and loosely piled. Such a method is good engineering where stone is available and of small cost.

GEORGE L. DILLMAN,* M. AM. SOC. C. E. (by letter).—This slide seems to have been an ordinary one, with motion extending over years, typical symptoms, springs, etc. The cause is always water. The cure is only drainage. It does not make any difference whether the slip is a small one at Natchez, Miss., larger ones at Culebra, or immense ones at Cow Creek, Tillamook, and the Frazier River. The Pacific Coast is dotted with them, to any one who has his eyes open. Mr. Dillman.

(1) These slides are natural and frequently overlooked by "engineers." The reservoir slide at Portland, Ore., so ably described by D. D. Clarke, M. Am. Soc. C. E., in two papers† before the Society, was such a case.

(2) They are imminent and precipitated by artificial means, such as the Berkeley slide, where a frog pond was a contributing cause and its abandonment the cure.

(3) They are caused wholly by constructions, such as the failure of the Calaveras Dam, the basis of the paper by Allen Hazen, M. Am. Soc. C. E., on "Hydraulic-Fill Dams."‡

These are all slides, and they are all caused by hydraulic pressure. The film of water only has to be thick enough to be continuous in order to act as myriads of jack-screws to lift and crowd the material in front. The resistance is variable—weight and friction—both being diminished by the presence of water, pressure to lift, and water lubrication. The force tending to move is variable, increasing with head until stability is overcome and motion ensues.

Drainage will cure any slide. Any drainage will lessen motion, and sufficient drainage will stop it. All that is necessary is to let the water get away a little more easily (and, therefore, more quickly) than it gets in. That is the principle. Tiling, perforated wooden boxes, drains filled with slag or other material to keep them drains, are only details. Probably the taking away of material as it comes is the most

* San Francisco, Cal.

† "A Phenomenal Land Slide," *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 322, and "A Phenomenal Land Slide—Supplement", *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 767.

‡ *Proceedings*, Am. Soc. C. E., April, 1920, p. 525.

Mr. Dillman. expensive cure, but it has been done. With superficial slides it may sometimes be cheaper than drainage.

Mr. Smith's paper is well written. It is good to call the attention of engineers to specific facts. If they can see through the incidents to the principles behind them, that is education. That is what these papers are for.

Mr. Smith. C. E. Smith,* M. Am. Soc. C. E. (by letter).—An inspection of the site of the box factory in August, 1920, several months after the completion of the drainage work, shows that no earth movements of any kind have occurred since the drains were constructed. Shortly after the slides were stopped, the factory was put on a production basis and is now being operated profitably.

Mr. Dillman is right in saying that it is good to call the attention of engineers to specific facts. It is to be hoped that they can see through the incidents to the principles behind them, which is much to be desired.

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* St. Louis, Mo.

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Paper No. 1453.

THE PROSPECTIVE COMPETITOR METHOD OF VALUATION OF PROPERTY

By M. L. Byers, M. Am. Soc. C. E.

With Discussion by Messrs. J. P. Snow, C. E. Grunsky, Joseph Mayer, J. E. Willoughby, R. W. Hawley, and M. L. Byers.

Synopsis.

The Courts having ruled that, in estimating the fair value of any operating property, the fact that the property is a going concern must be taken into consideration and that this fact is an element of value, there remains to be devised a method whereby, when the fairness of the rates is in question, this fair value of the property can be estimated with reasonable accuracy.

The method which of late years has found most favor is that usually termed the Cost-of-Reproduction Method. In a general way, value is arrived at by this method about as follows:

From cost of reproduction of the physical property (including a percentage for overhead costs) deduct "depreciation"; to this add going concern value.

The method does not seem to rest on sound economic principles, and parts of the practical operations involved in its use do not appeal to the logical mind. Too much is left to mere personal opinion uncontrolled by guiding principles.

One Court observed that he reluctantly accepted this method because he did not know of a better one. The remark of the Master

in the recent Denver Water Company Case with reference to going concern value is also illustrative. He states:

"There is no absolute standard by which the fair value of this element can be determined, and I adopt $800 000, because no matter how often I have considered the evidence and the arguments my mind always comes back to this amount as reasonable and fair to all the parties."

Further illustration of the difficulties produced by this method is found in the Minnesota Rate Decisions, in which Justice Hughes pointed out in connection with the valuation of the land of the railway that "the assumption of its [the railway's] non-existence, and at the same time that the values that rest upon it remain unchanged, is impossible and cannot be entertained."

Presumably the reproduction theory may be based on the view that this presents the means of estimating what the property under valuation would have cost to construct and develop if present-day conditions as to prices of labor, material, land, etc., density of population, and similar elements had been present during such period of construction and development. This is merely a variation of the theory that the fair value of the property is measured by the investment, whereas, as a matter of economic fact, there is, in general, no necessary relation whatever between the value and the cost of a property.

What is urgently needed at the present time is a rational method of valuation which is in harmony with sound economic principles and does not clash unnecessarily with well-established legal decisions.

The analysis of a new method of determining the fair value of a railroad property, which follows, is founded on seven basic propositions:

1.—A railroad corporation is a private corporation; its property, although devoted to the service of the public, is private property, and its value is as completely under the protection of the Constitution as that of any other private property.

2.—The property of a railroad corporation devoted to the service of the public, and which has been acquired through gifts or donations, or from earnings, is, equally with property acquired from the proceeds of the sale of securities, private property, and as such is under the protection of the Constitution. The surplus from the earnings of the corporation is likewise private property.

3.—Every enterprise subject to regulation is entitled to be permitted to earn such a return as, in the field of free and open competition, would ordinarily accrue to a similar expenditure of energy, foresight, and capital.

4.—A public utility is not entitled to a return accruable solely because of its possession of monopolistic protection from competition.

5.—Every kind of business, without any exception, has an element of value known as going concern value, and such going value is in no way connected with the monopoly or good will value.

6.—The market value of any property results from the use to which it is put and varies with the profitableness of that use, present and prospective, actual and anticipated. There is no pecuniary value outside of that which results from such use. The amount and profitable character of such use determine the value. The measure of that value is its profitableness, present and prospective, actual and anticipated.

7.—The fair value of a property is measured by its profitableness, present and prospective, actual and anticipated, under fair rates.

The analysis results in the development of the Prospective Competitor Method of valuation. The steps recommended in the case of a valuation of a monopoly by this method are as follows:

1.—Production of a prospective competitor (instead of "reproduction" of the property under valuation).

2.—Determination of the rate level which (within that maximum period of time beyond which capital would not be willing to wait) will enable said competitor to earn a fair return on its investment to date.

3.—Determination of the annual net earnings, as of the date of valuation, of the property under valuation and at the rate level indicated in Item (2).

4.—Capitalization of the annual net earnings indicated in Item (3); this capitalized amount is the fair value of the property under valuation. In the case of properties subject to competition, the fair value of all such properties combined (and which combination, consequently, may be treated as a

monopoly) is first obtained by the preceding method; the common fair rate level is also ascertained. By the application of this rate level to the individual properties, the fair return of each is ascertained. Capitalization of such individual fair return gives the individual fair value.

The Competitor Method practically involves no difficulties which are not met with in the Reproduction Method, while it is logical throughout and is based on sound principles of economics.

I. General Principles and Recommended Procedure.

1.—Railway Valuation, Its Theory and Uses.

For many years the railways have contended that the activities of the Interstate Commerce Commission and of the State Commissions have resulted in the lowering of transportation rates to a point where the prosperity of the industry has been seriously interfered with. The Commissions have always replied to this accusation that, while the railroads certainly are not earning the normal rate of return on their capitalization, they are earning, at the very least, a fair rate of return on their fair values; that the real trouble with the railroads is and has been that they are so greatly over-capitalized that a fair charge for transportation produces a comparatively small rate of return on the enormously watered capitalization.

After many years of persistent recommendation on the part of the Interstate Commerce Commission, Congress passed the law calling for valuation of the railroads, which is now in progress. The principal reasons given by the Commission for its recommendation were that the information was needed in connection with the proper execution of its duties, and that the controversy over railway values should finally be cleared up in the interest of all parties.

Since the passage of the Valuation Act, numerous comments have been heard at frequent intervals, among these being statements that it is impossible to find the value of a railway property; that the value, being dependent on earnings, is constantly changing, and that the valuation to-day would be out of date to-morrow; that valuation is useless for rate-making purposes, as the rates do not depend in any

way on the value of the property used; that there is no need for the regulation of railway rates, for the reason that the business itself affords a sufficient check over excessive rates (the very great general increase instituted by the Government as a war measure hardly supports this last contention); and that there are a number of values for the same property, depending on the use to which the data are to be put.

Discussion as to what constitutes fair value has been even more interesting, being largely colored by the interests of those entering into such discussion. The representatives of the States have been consistent in their position that it was their duty to see that the public did not have imposed on it a valuation entirely in excess of the equities of the situation. The railroads have been equally energetic in their efforts to see that the resulting valuation did not arrive at an inequitably small result. Apparently, the Division of Valuation of the Interstate Commerce Commission early came to the conclusion that the railroads were amply able, on all proper occasions, to demonstrate clearly the propriety of deciding in their favor any doubtful points, and that the proper policy was to start out with assumptions which would produce the smallest values; then, through hearings before the Division and before the Interstate Commerce Commission, thrash out the points of controversy to an equitable conclusion.

One of the first claims made as to the constitution of "value" was that it was identical with "cost of reproduction" of the existing property, no allowance being made for appreciation in the value of land; the railroads, being public utilities, were assumed not to be entitled to any unearned increment. As this position apparently became legally untenable, and as the items of cost of reproduction ordinarily overlooked by the casual observer became increasingly important, a tendency developed to assert that value equals, not cost of reproduction, but original cost to date of the existing properties.

The effect on value produced by depreciation and appreciation existing in the property has also been widely discussed. Another of the frequently heard statements in connection with the ascertainment of value is that it is not a matter of fixed rules, this quotation from a decision of the Supreme Court being interpreted to the effect that there are no fundamental principles on which the ascertainment of value can be based, thus still further befogging an already complex subject.

In none of the many discussions on the ascertainment of value has there been any complete analysis of the problem, followed by an attempt to work out principles solidly founded on equity and economics. The present discussion is an attempt to analyze the entire problem and then to build up a rational theory of valuation in harmony, if possible, with such legal generalities as so far have been promulgated.

2.—What Is a Public Utility?

Ordinarily, when a public utility is mentioned, it is understood that a utility supplying water, or gas, or electric light, or transportation to a community, and operating under a charter as a corporation, is meant; that other activities such as farming, manufacture of steel, clothing, food, etc., are not so regarded.

There has gradually developed in the public mind a feeling that public utilities, because of their being public utilities, are subject to entirely different economic laws and have entirely different legal and moral rights and obligations from those of other forms of commercial activity. This unreasoned feeling has been the cause of much misunderstanding and dissatisfaction and of much ill-advised regulation.

Examined from the broad point of view, almost every form of commercial activity is a public utility. The farmer does not raise the greater portion of his crops for his own use, but for the use of the general public. The manufacturer manufactures his product with the same object in view. The essential difference between the so-called public utilities and other forms of enterprise lies in the fact that, in the case of the public utility, there usually enters the element of monopoly, either because of the physical conditions surrounding it or because it is to the interest of the public that the enterprise shall be conducted without competition.

Perhaps a broad definition for a public utility is that it is any industrial activity conducted on such a large scale or under such conditions that the interests of a considerable portion of the containing community are vitally affected thereby.

Looking to the future—bearing in mind the great strides which have been made during the past fifty years in the organization of various enterprises—it seems probable that the time is not far distant when many other enterprises will have arrived at that perfection of organiza-

tion where sufficient control is exercised by a few persons to enable them, if unrestrained, to reap the rewards possible only to monopoly.

It is sometimes stated that an enterprise conducted by a private individual is not subject to Government regulation. This is simply a misstatement of fact. It is also frequently claimed that the giving of the right of eminent domain by the public to certain corporations introduces factors which justify the public in establishing regulations not otherwise equitable. The purpose of the giving of the right of eminent domain has been widely misunderstood and misrepresented. In certain types of industry, in the absence of the right of eminent domain, it would be possible for a single individual to obstruct, or even to prevent entirely, the development of a particular enterprise. Yet it has often been to the public interest to induce private capital to enter such fields. To this end, under various restrictions, the right to condemn property on payment of fair compensation has been granted. To claim that the conferring of this right carries with it the privilege of making later exactions on the grantee is as if a householder employed an artisan to do certain work for him in his house, gave him the key to the house in order to enable him to enter therein, and then stated to the artisan that because of having given him the key he is entitled to pay him a smaller compensation than the going rate for such service.

3.—When and Why Regulation of Industrial Activity Becomes Desirable.

Ordinarily, in industrial activity on a considerable scale, there are four important parties at interest, namely, the owner of the activity (the stockholder), the employee, the individual user of the product, and the general public.

Simply in his capacity of owner, the stockholder is interested only in the amount of profit obtainable from his investment, although that interest extends to the broad point of view and is not confined solely to the narrow limit of immediate results.

The employee is primarily interested in the conditions of service under which he is employed and the compensation received for such service.

The individual user of the service is interested in its quality and in the payment which he must make therefor. If the service which he desires is an unusually expensive one, it is to his interest that pay-

ment for such service be based on general averages of cost, rather than on the cost of the individual service itself. For example, the man who lives at one end of the street car line and has his business at the other end, desires that a single average fare be collected from each passenger.

The general public, although less acutely interested than the three other parties, does have a broad general interest of considerable importance. It is to its interest that the relation between productivity and effort expended (such as the amount of capital utilized) in securing the total production, be as favorable as possible, for by improving this ratio the wealth of the community can be increased. The basis of the division of cost of service is also important. Shall the user pay the entire cost of production of that which he uses or shall a portion of this cost be distributed (as by taxes) to the general public? In this connection, the general public must bear in mind that there is frequently an indirect value of the service which it alone receives; for example, the first transcontinental railroad in the United States made possible the development of the Western territories and the knitting together of widely separated populations into a homogeneous community. Lastly, there remains the relation between the character of the industrial activities and the requirement for good government. Will a given policy of industrial activity, if permitted, react on the political security of the community?

Ordinarily, competition and the relatively diminutive character of the individual industrial activity have been found to regulate satisfactorily the relations of the four interested parties mentioned. In the absence of competition, whether due to physical conditions or otherwise, and even in the presence of competition in the case of activities of great importance, some form of artificial regulation of the more or less conflicting interests of the various parties must be resorted to, or abuses will creep in and multiply to the point where dangerous unrest is created, as has been the case in several localities and on several occasions in the transportation industry of the United States.

4.—What Is Valuation and What Are Its Uses in Connection with Industrial Regulation?

The casual observer would be apt to say that valuation is the process of finding value. Such, however, is far from being the entire story.

Valuation is the process of ascertaining one or more of a number of things, among which the following may be mentioned as being the most prominent:

1.—Forced sale value—being the amount in dollars for which a property would sell if thrown on the market, as at sheriff sale.

2.—Market value—being the amount for which a property would sell when the owner is desirous, but not compelled to sell, and when the purchaser is desirous, but not compelled to purchase.

3.—Fair value—being the value for which the property would sell if the commodity rates were fair, if the owner was desirous, but not obliged to sell, and if the buyer was desirous, but not obliged to buy.

4.—Investment to date—being the net amount of capital which has been invested in the property to date, and, consequently, including expenditures for abandoned property and excluding the value of aids, gifts, grants, and donations received.

5.—Original cost to date—being the same as investment to date, except that the expenditure for abandoned property is excluded.

6.—Cost of reproduction—being the estimated cost, under normal conditions as of the date of valuation, of reproducing the existing property.

7.—Physical depreciation—being the percentage of the useful life of the existing property which has been consumed through age and use.

8.—Depreciation of obsolescence and inadequacy—being the percentage of the life of the property, as a whole and for its present uses, which has elapsed.

9.—Tax value—being the amount on which the property is taxed (and depending on the local tax laws).

The principal uses claimed for valuation in connection with industrial regulation are that the data are more or less necessary in order to arrive at equitable conclusions with reference to taxation, to individual rates, to the general rate level, to the amount to be paid in cases of condemnation and sale, to capitalization, and to the giving of needed information to the investor.

5.—What Is Value and How Many Kinds of Value Are There?

It is the fashion in certain circles to speak of value as being a nebulous sort of thing, undefinable and incomprehensible for valuation purposes. It is another example of the cuttle-fish instinct to muddy the water with which the subject of valuation is so frequently approached. The necessary first step in any logical consideration of the subject of valuation is to define clearly this term in its intended uses.

Definition.—Value is a ratio in exchange.

In other words, it is a statement of the number of units of one kind for which a single unit of another kind is exchangeable. It is a mere abstraction and requires the addition of an explanatory and limiting term to make it available for practical use. To illustrate, a diamond may have the same money or pecuniary value as fifty tons of coal, but it has not the heating value of one ton.

The forced-sale value of a property (which is usually adopted as the tax value) is often only half, or less, of the normal fair value where neither buyer nor seller act under compulsion. Also, there is the sentimental value which the owner may place on a commercially worthless heirloom.

Definition.—Pecuniary value is the amount of money for which a property will sell when the owner desires, but is not obliged to sell, and the purchaser desires, but is not obliged to buy.

In this discussion, unless specifically stated to the contrary, it is this value that is referred to whenever the term "value" is used.

Evidently (pecuniary) value, as defined, is determined solely by economic considerations.

Definition.—Market value is the value of a property as determined by present and probable future economic conditions.

Regardless of the fairness of the rates in effect in any property subject to regulation, market value is determined by the existence and probable continuance or discontinuance of these rates, and in disregard of their fairness except as this may reflect on their probable permanence.

Definition.—Fair value is the value which a property would have if, for the present rates, fair rates were substituted, with the reasonable certainty that they would continue in the future.

It is evident that there are as many values for a property as there are points of view from which the subject can be approached. For the

purpose of this discussion, value will be limited to that for rate regulation and for condemnation and sale.

6.—Profitableness Under Fair Rates, the Measure of Fair Value.

What is undoubtedly the fundamental economic law to be kept in mind in the valuation of property for condemnation or for rate regulation has been stated by the Supreme Court of the United States, in Cleveland, Cincinnati, Chicago and St. Louis Railway Company *vs.* Backus, 154 U. S. 439, in the following terms:

"But the value of the property results from the use to which it is put and varies with the profitableness of that use, present and prospective, actual and anticipated. There is no pecuniary value outside of that which results from such use. The amount and profitable character of such use determines the value. * * *"

Although it is true that, for a property not subject to regulation, the value at any given time is measured by the profitableness, present and prospective, actual and anticipated, this is not the value (more properly termed the fair value) to be found for use in condemnation or for use with reference to rate regulation. For such purpose, fair value is measured by profitableness, present and prospective, actual and anticipated, under fair rates. If this was not the case, it would be possible, through rate regulation, to reduce the rates and, consequently, the net earnings and the profitableness of the property to any point arbitrarily selected, and then obtain a value for condemnation purposes, based on such unfair rates, which would be far below the fair value of the property. Such a procedure would seem, equitably, to be inconceivable.

The Courts have called attention to the fact that, where the fairness of rates is in doubt, profitableness resulting from such rates cannot properly be used in the determination of fair value. This is axiomatic. The converse is also axiomatic, namely, that where the rates are acknowledged to be fair, the fair value can properly be ascertained by the use of the resulting profitableness as its measure.

As fair value is dependent on fair rates and fair rates on fair value, it would seem immaterial, when neither is known, which is to be ascertained first, except that, contrary to the general view, it is practically much more convenient first to ascertain fair rates and then therefrom to ascertain fair value, than to proceed in the opposite direction.

7.—"Cost" *vs.* Value.

When the fairness of the rates is questioned, it is impossible directly to apply any known variety of profitableness to the measurement of fair value; consequently, "cost" in some relation to the property must be used. A brief consideration of several special cases, however, clearly indicates that, in general, there is no necessary fixed relation between the cost of a property and either its market value or its fair value.*

If, therefore, cost is to be used in the determination of fair value, it is necessary that it be applied (1), under such special conditions as shall create a known relation between such cost and the fair value, and (2), where such established relation shall be equitable. Then, wherever such conditions can be simulated, one can properly utilize such variety of cost in the determination of fair value.

Except through occasional accidental coincidence, however, the only circumstances under which cost bears a fixed relation to value are those in which some one having the necessary power decrees that such commodity rates shall be charged that the resultant profitableness will establish this chosen relationship. This can be done in the case of a monopoly, but it is an impossibility in the case of two or more properties subject to competition.†

In order that the owner of a monopoly under regulation shall not be deprived of that protection against confiscation which is guaranteed by the Constitution, the next step is to inquire in what specific case, capable of being simulated under the above circumstances, it is equitable to enforce some relation between cost and value and as to just what this cost and this relation are. This is a problem in economics and economic principles only are involved.

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* A plant for the manufacture of furniture is constructed in the wilderness at a cost of $100 000. No purchasers can be found for the product and the salvage value of the plant is only $1 000. Here the relation of cost to value is as 100 to 1. A similar plant is built at the same cost where labor and raw materials are plentiful and where the market is so good that the average profits are $18 000 per year. If capital is worth 6% per annum, the value of this plant is $300 000 and the relation of cost to value is as 1 to 3. Between these extremes lie all varieties of relations between cost and value.

† Consider two plants, A and B, in competition. Being in competition, they must adopt the same selling prices, or one would do all the business. Assume that the "cost" of A is $100 000 and that of B $200 000; also that, at the present selling prices, A's gross earnings are $6 000, and his net earnings are $4 000 per year, while B's gross is $9 000 and his net $5 000. Suppose it is decreed that the selling price shall be raised one-third in order that A's net shall become $6 000. Then B's net becomes $8 000. Capitalizing, at 6%, A's value becomes $\frac{6\,000}{0.06} = \$100\,000$ and equals his "cost", while B's value becomes $\frac{8\,000}{0.06} = \$133\,333$ as against a "cost" of $200 000. No selling price applied to both will give each the same ratio of cost to value.

The owner of every regulated enterprise is entitled to earn the same profit as the same amount of capital, intelligence, and industry would normally enable him to earn in the field of free and open competition.* Therefore, it becomes of fundamental importance to establish the principles under which commodity selling price is determined in the open field of free competition.

8.—The Principles Governing Free Competition—Their Application to Valuation of a Monopoly.

Let us suppose some community is served as to some commodity by a producer whom we will call briefly the A. and B. Company. This company has been in the field for years and has no competition. Local conditions do not prevent competition, and the commodity price is not limited by the public's ability to pay.

Certain capitalists, whom we will call the X. Y. Company, are looking for a favorable opportunity to make investment of their surplus capital. They, of course, have the range of the entire field of investment, regulated and unregulated, foreign and domestic, from which to choose; they will enter that field which, on investigation, gives promise of offering the greatest inducement as to profit, risk considered. Among others, the field occupied by the A. and B. Company comes under their consideration. Their decision as to whether or not to become competitors of the A. and B. Company will be governed by the answer which they accept to the following question:

"Considering the difficulty of establishing a new enterprise in competition with an old, established concern, will it be possible, within a reasonable period of time for such new concern to create a more than normally profitable business, considering the amount of capital which would have to be invested?"

As to the amount of capital necessary to be invested in the proposed new enterprise, the X. Y. Company realizes the following:

1.—Neither the amount of money which it was necessary for the A. and B. Company to invest in its plant, nor the profitableness of that enterprise, has any bearing on the situation.

* To deny this is to establish one of two conditions: If a greater return is allowed, the public is assessed a greater than normal charge for service rendered. If the right to earn at least this amount is denied, and it becomes known that such is the rule, capital cannot be enticed into the enterprise, and it will become dependent on Government ownership or will be destroyed. For it is an unquestionable fact that liquid capital is free and will be invested only where it can obtain at least the normal rate of return, risk considered. The entire field of investment in freely competitive enterprises, at home and abroad, is open to it at all times. It cannot be coerced into entering a regulated field where it will be compelled to accept less than the normal return obtainable in the competitive field.

2.—The X. Y. Company will be obliged to pay present-day prices for the land, labor, and material entering into the construction of its plant.

3.—The possible volume of business is limited in amount; also this total available must be divided with the A. and B. Company, already in the field.

4.—The A. and B. Company cannot secure commodity prices higher than those which the customer can afford to pay; the X. Y. Company cannot secure prices higher than those of the A. and B. Company.

5.—In determining the total amount of capital which the X. Y. Company must invest in its proposed enterprise, there must be included not only the actual expenditures for construction, but also the loss of a normal rate of return on its expenditures during the construction period and during the lean years (the "development period") which must follow, during which its volume of business is being built up to a sufficient density to enable the normal rate of return to be earned on the entire investment.

One of the most common causes of failure of business concerns is insufficient capital; the insufficiency is all too frequently caused by under-estimating the capital requirements of the development period and by failing to provide adequately for contingencies.

In determining the amount of capital necessary in the competition with the A. and B. Company, the X. Y. Company would ordinarily prepare its own plans, estimate the purchase price of the land prospectively required, and estimate the construction and operating costs of the prospective plant, etc., in addition to estimating the volume of business and the unit selling price probably obtainable.

As a matter of convenience, however, the X. Y. Company might conclude that it could obtain practically the same construction-cost and operating-cost figures by taking the estimated cost of reproduction of the A. and B. Company's plant under present-day conditions and by using the A. and B. Company's unit operating costs. Herein lies the sole economic justification for accepting "cost of reproduction" as a major element in the determination of value. The X. Y. Company would still be obliged, however, to make an independent estimate, not only of the amount of business which it, the X. Y. Company, as a competitor, could secure, but also of the capital cost to it of its develop-

ment period under these conditions; it could not substitute A. and B. Company data for this purpose and obtain acceptable results.

In the field of free competition, it is the conditions facing the prospective competitor and not the cost of the existing property which control commodity selling price and, consequently, profitableness and value of the latter. The value of any unregulated individual enterprise bears no necessary relation to its cost. For illustration: The cost of a manufacturing plant constructed in the heart of Alaska would be much greater than that of a duplicate plant constructed on a railroad in the State of New York; but the profitableness of the New York plant, and, consequently, its value, would probably be much greater than that of the Alaska plant.

Consideration of the preceding illustration of the working of competition in the regulation of commodity price indicates as follows:

A.—The fair commodity selling price at the time of investigation and in the field occupied by the A. and B. Company is that which is just sufficient to permit a prospective competitor—as the X. Y. Company—a normal rate of return, risk considered, on its necessary capital outlay, this being based on a development period not exceeding that reasonable time which capital would be content to wait for a fair return. For this is a free field, wide open to all competition; this commodity price neither invites competition nor repels it; a higher price established by the A. and B. Company would attract competition, while a lower price would repel it.

B.—The measure of fair value of the A. and B. Company is its profitableness, present and prospective, at this fair commodity selling price—just sufficient to permit a prospective competitor to earn a normal rate of return, risk considered, on its necessary capital outlay.

In valuation in the non-competitive fields, it is proper, therefore, instead of constructing a theoretical "reproduced property" as is sometimes suggested, to construct a theoretical "prospective competitor". Thereby the conditions governing commodity price in the competitive fields are simulated and the way is thus opened to estimate fair commodity price and fair value of the property under valuation.

It is to be noted that the illustration, and, consequently, the principles deduced therefrom, refer only to the case where the existing company has no competitor; in other words, it is a monopoly, though

able to remain so only by not charging unfair prices for its commodities.

9.—Method of Estimating the Fair Value of an Enterprise Subject to Competition.

Where the valuation of competitive properties is at issue, the same principles can be applied by first considering all the competitive properties as a single unit, such unit being, of course, a monopoly of the type used in the illustration. In this manner, through resort to the use of the prospective competitor for this monopoly, the fair commodity price and the sum of the fair values of all the competitors are obtained.

The problem still to be solved is, having estimated the sum of the fair profitableness and of the fair values of the properties regarded as a unit, and having records showing their present and past individual operating statistics, how to find their individual fair values.

The first step is to compare the sum of the fair profitableness with the sum of the normal actual profitableness and ascertain the percentage by which the actual must be raised or lowered to equal the fair profitableness. Next, apply this percentage as a correction to each individual normal actual profitableness in order to obtain its fair profitableness; from this, compute fair value by capitalizing at the normal rate of return, risk considered, in such industry. For example, three railroads—A, B, and C—are in competition in a certain district; they have present gross earnings of $10 000 000 and net earnings of $4 000 000, divided between them, as shown in Table 1.

TABLE 1.

Road. (1)	Actual gross. (2)	Actual net. (3)	Fair net. (4)	Fair value at 6 per cent. (5)
A	$2 000 000	$800 000	$1 000 000	$16 667 000
B	3 000 000	950 000	1 250 000	20 833 000
C	5 000 000	2 250 000	2 750 000	45 833 000
Total.....	$10 000 000	$4 000 000	$5 000 000	$83 333 000

Assume that valuation indicated that the total fair profitableness of the three railroads should be $5 000 000 instead of $4 000 000. In order to increase the net of $4 000 000 to $5 000 000, it will be necessary to

increase the gross earnings $1 000 000, or 10%, this being brought about by a 10% raise in the general rate level. Adding 10% of its gross earnings to the present net earnings of each railroad gives the fair net, as shown in Table 1, Column 4. This amount capitalized at 6% indicates a fair value, as shown in Column 5. If an individual rate, being too high, is lowered, this merely requires that slightly more than 10% be added to the other rates to produce the necessary fair gross earnings.

10.—Assumptions and Mode of Procedure in the Determination of the Fair Value of a Monopoly Railroad.

In devising the various assumptions which must be adopted, in the "competitor" method of estimating fair value, two fundamental facts should be kept constantly in view as a guide, namely:

1.—The object of the resort to the theoretical "prospective competitor" is to simulate, for the property under valuation, the condition which would normally be produced if that monopoly were actually subject to normal free competition, and thereby to make possible an estimate of what its profitableness, under such conditions, would be.

2.—It is evident that the entire economic justification for the use of cost of reproduction as one element in the determination of value lies in that, if properly used, it ordinarily gives practically the same results as would be obtained from an out-and-out new estimate of the cost of construction of a prospective competitive plant; the assumption must, consequently, always be made with this fact in view.

A.—Assumptions.—

Assumption 1.—The prospective competitor has the same accessibility to all sources of revenue as the property under valuation. In other words, it can, when properly seasoned, offer to the customers of the monopoly exactly the same facilities as those offered by the monopoly. To illustrate, the company under valuation has certain traffic agreements with its connections; the prospective competitor must be assumed, at the cost of a reasonable amount of negotiation, to secure the same terms. The old company reaches a great manufacturing plant by means of a track located through the only available route; the competitor must be assumed to be able to reach the same plant at "cost of reproduction" of the old company's track.

Assumption 2.—The development of the earning power of the competitor will occur through competition, under normal conditions, with the monopoly under valuation, for all the available business, present and prospective.

Inasmuch as the competitor, when properly seasoned, is able to offer the same inducements as can be offered by the property under valuation, it would ordinarily appear that, shortly after the commencement of its operation, the competitor should divide equally with the property under valuation the then business of the latter. Thereafter, the normal rate of growth of business in the industry, in recent years, is the rate of growth to be assumed for the business of the competitor, for the reason that one of the objects to be accomplished is the reduction of development cost to a normal basis regardless of the history of the individual enterprise (usually a history of small enterprises, failing, re-organized, combined, and re-combined over long periods of time).

Assumption 3.—The construction of the competitor's plant will be carried out under present-day methods, at present-day normal prices, and under present-day legal and economic conditions. The dates of construction of the individual pieces of property will be selected so as economically to harmonize with the needs indicated by the programme of development of its earning power.

Except as can clearly be shown would be carried out otherwise under the present-day conditions confronting the competitor, the physical characteristics, and, consequently, the construction quantities, including land, of the competitor are taken as ultimately identical with those of the monopoly under valuation. Construction evidently economically greatly in excess of requirements would be developed piecemeal as required. Thus, the competitor railroad would ultimately have the same length of line, grades, curvature, quantities of grading, etc., as the railroad under valuation, but the structure built of materials no longer available would be reproduced with accessible materials of equal suitability.

Assumption 4.—The naked land value of the lands adjacent to those required by the competitor will be the same, in each case, as the naked land value of the lands adjacent to those of the monopoly. The improvements assumed to be resting on the land required for the competitor are of the same character and value per acre as those on similar lands in the general vicinity, but not necessarily adjoining the monop-

oly's local facilities. The right of a competitor to use public streets would, or would not, be paid for, according as present-day practices in the community dictate.

Assumption 5.—The competitor will be obliged to pay for its land whatever amounts experience teaches would ordinarily have to be paid, in such communities, by railroads, for lands having naked land values equal to those surrounding the lands of the old company. These prices would include payments account severance and other damages.

B.—Mode of Procedure.—

1.—A programme showing in commodities, not in dollars, the estimated gross business of the competitor, year by year, will be developed for a period sufficiently long to equal, at its end, the normal business of the property under valuation, as of the date of valuation.

2.—A construction cost programme based, in general, on the reproduction of the existing properties of the monopoly under valuation, but at a rate and in an order harmonizing economically with the adopted traffic programme of the competitor, will next be made.

3.—An operating cost programme showing year by year the competitor's estimated gross operating costs, will next be constructed. There must be included in operating expenses a proper annual allowance for deterioration, obsolescence, abandoned property, maintenance, taxes, and rentals accruing after the commencement of operation; amounts accruing before this date are a proper addition to construction costs.

4.—Tables of annual gross and net earnings of the competitor should next be prepared. The rate level to be used in estimating the competitor's gross earnings is the lowest rate level which, within that reasonable time which capital would be contented to wait, will produce net earnings for the competitor equal to a fair return on the competitor's cost to date.

5.—From the preceding, the final table showing the competitor's estimated cost to date for each year and including loss of interest during construction and loss of fair return during operation can be prepared.

6.—Having found the proper general rate level for use in estimating the prospective competitor's gross earnings, the fair value of the property under valuation is obtained by capitalizing its profitableness at this same rate level.

II. General Discussion of Miscellaneous Subjects.

1. Summary of Important Principles Enunciated by the Courts with Reference to Fair Value and Fair Return.

The essential principles enunciated by the Courts with reference to fair value and fair return can be summarized as follows:

1.—What the company is entitled to ask in order that it may have just compensation, is a fair return on the fair or reasonable value of that which is at the time utilized for the public convenience.

2.—On the other hand, what the public is entitled to demand is that no more than a reasonable compensation shall be exacted for the service performed.

3.—The public does not underwrite unwise or improvident expenditures; such expenditures do not add to the fair or reasonable value of any property and the company cannot demand of the public that it be compensated for them.

We still are left by these decisions to discover the principles on which a proper method can be based for ascertaining (1), what is the fair value of the property of the company as used; and (2), what is a reasonable compensation for the service performed.

Nor does the following from San Diego Land and Town Company *vs.* Jasper, materially forward the investigation:

"The ascertainment of that value is not controlled by artificial rules. It is not a matter of formulas, but there must be a reasonable judgment having its basis in a proper consideration of all relevant facts. The scope of the inquiry was thus broadly described in Smyth *vs.* Ames, 'in order to ascertain that value, the original cost of construction, the amount expended in permanent improvements, the amount and market value of its bonds and stock, the present as compared with the original cost of construction, the probable earning capacity under particular rates prescribed by statute, and the sum required to meet operating expenses, are all matters for consideration, and are to be given such weight as may be just and right in each case. We do not say that there may not be other matters to be regarded in estimating the value of the property.'"

2.—That Reasonable Compensation to Which Rates Are to Be Limited.

Most enterprises under regulation differ from those operating in a free competitive field in that the former, to a greater or less extent, are

protected from competition by the interposition of various physical and legal barriers. Thus, a water-works may be protected from competition by legislative enactment, or it may be protected through ownership of the only reasonably available source of supply.

Reading between the lines of the Court decisions with reference to the fairness of individual cases of rate regulation, a broad guiding principle which seems to have been kept in mind is that the ruling should be such as to deprive the regulated enterprise of the benefits of any monopolistic protection and to permit of the earning only of such a return as, in the field of free and open competition, would ordinarily accrue to a similar expenditure of energy, foresight, and capital. This is well illustrated by the remark of Justice Holmes in Cedar Rapids Gas Company *vs.* Cedar Rapids, that what the Court excluded was good will or advantage incident to the possession of a monopoly so far as might be supposed to give the plaintiff the power to charge more than a reasonable price. The following may aid in illustration:

1. Assume that a certain farmer takes his wheat to Chicago and demands for it $3 per bushel. Is that an unreasonable price? No, not if he can get it, for this is a free, competitive market; if the circumstances did not justify the price, others would fill the demand at a lower price, compelling him also to lower his price or keep his wheat.

2. A certain community obtains its water supply from a lake 10 miles distant, hauling by wagon at a cost of $1 per 1 000 gal. The A. and B. Company builds a pipe line and delivers water to the town at a cost, including normal rate of profit, of 10 cents per 1 000 gal.; it charges 50 cents, making a very high return on the investment. The quantity of water consumed by the town is so small, however, that no rival entering the field and dividing the demand with the present company could furnish water profitably for less than 55 cents. Is the 50-cent rate unreasonable? No, because the business is freely competitive; as the demand increases, the company will be obliged to lower its rates to prevent competition. Meanwhile, the company, by its enterprise, is performing a public service by saving to the town 50 cents on each 1 000 gal. consumed as compared with the cost under the old method of haul.

3. Suppose that, in the preceding case, the A. and B. Company buys the water supply and thus renders competition difficult, as the next source of supply is 20 miles away. Because of the distance—20 miles—

and the small demand of the town, it would cost the X. Y. Company 80 cents per 1 000 gal. to deliver water at a normal profit. Would it be reasonable for the A. and B. Company to charge 75 cents under these circumstances? The charge would be unreasonable because it is now based, not on that which would be established if competition prevailed, but, on the contrary, on the holding of a monopoly of the water supply in the vicinity.

4. A railroad was built from the City of A into certain undeveloped coalfields at B; *en route* it passes through a gorge which furnishes the only low-grade line across the mountain between the two points. As it completely occupies the gorge, no other railroad can be built, except at a prohibitive cost, over the mountain. A transportation charge which would net 6% on the cost of any line over the mountain would pay 20% on the cost of the gorge route. Owing to the absence of other coalfields in the vicinity, coal could be moved at a profit even at this high charge. Would the charge under these conditions be unfair? The charge would be unfair because it is based on a monopoly of the gorge route, and not on rates which competition would establish in the absence of this physical barrier.

The frequently recurring statement in the older decisions that "a public service company cannot lawfully charge more than the services are reasonably worth to the public as individuals" becomes an absurdity if literally interpreted. Suppose, that, for example, the market for lumber on the A. and B. Railroad is at B and is $10 per 1 000 ft.; a farmer owns timber some miles from the railroad at A, its distance from the railroad being so great that the cost of cutting and hauling to the railroad is $10 per 1 000 ft., being just equal to the price which he would receive for the timber if delivered to the market at B. Inasmuch as there would be no profit to him in the transaction, the value to him of the service performed by the railroad company in hauling his timber from A to B would be nothing; consequently, under the literal interpretation of the decision, the railroad company could make no charge for the service, because it possessed no value to the owner of the shipment.

In the later decisions of the Courts, it has evidently been recognized that rates, to be fair, must be fair to both parties to the transaction, and that the statement as to the value of the service is meaningless; it is consequently omitted. Thus, in Darnell *vs.* Edwards, 244 U. S.

564, the Court, while pointing out that the public should not be compelled to pay rates based on extravagant expenditures, states: "In determining whether rates are confiscatory, because not yielding a proper return, the basis of calculation is the fair value of the property used in the service of the public."

3.—The Fair Value of a Property Subject to Rate Regulation Is the Same for Condemnation as for Rate Regulation.

The well-known principle of law to be observed in condemnation cases is that the fair price to be paid the owner is that on which a buyer, desiring but not compelled to buy, would agree with a seller, desiring but not compelled to sell. The basis of agreement between the buyer and the seller, in such cases, is a conscious or unconscious meeting of their minds as to the profitableness, present and prospective, actual and anticipated, of the property under discussion and a translation of this into terms of value.

Where the property is subject to rate regulation and where, therefore, the profitableness can be varied by regulating the rates, it is evident that this rule must be modified so that it calls for a meeting of the minds of the prospective buyer and the prospective seller, based on their views of profitableness under fair rates.

In case of rate regulation, the owner is entitled to such rates as will enable him to earn a fair return on the fair value of his property. The measure of the fair value of his property is evidently the profitableness under fair rates, but this is identical with the fair value as determined by the previously mentioned rule to be observed in condemnation where rate regulation is in effect.

It is assumed, of course, that the same property is the subject of consideration in each case and that, if only the portion of the property used for carrier purposes is considered for rate regulation, the same (and not also all the non-carrier property) will be the subject of consideration for condemnation.

4.—The General Rate Level (and not Individual Rates) Is the Subject of the Inquiry in the Determination of Fair Value of a Railway Property.

The movement of bituminous coal to the Lake ports furnishes an excellent illustration of the situation to be dealt with in the consid-

eration of individual rates. Coals for the Lake ports are obtained from the coalfields of Ohio, Indiana, Illinois, West Virginia, and Kentucky. The length of haul varies from a few miles to several hundred miles; also, the cost of transportation per ton-mile varies because of variations in grades, size of engine, etc. The quality of the coal, and, consequently, the selling price at the Lake port, varies with the different fields. The cost of production of the coal at the mines varies because of differences in rate of pay of the miners, thickness of vein, relative quantity of slate, etc. A uniform rate per ton-mile would drive some of the more distant fields out of the market. The situation resolves itself into this, that, with a fixed coal selling price per ton at the Lake port, the profit on the total operation involved is to be divided between the owner of the coal, the coal miner, the coal operator, the transporter, and the distributor, on some equitable basis.

It is evident that the position taken by so many of those engaged in the regulation of rates, to the effect that the fair value of the property used is not the principal factor and is often not even an important factor in the determination of fair rates, is entirely correct when limited to the consideration of individual rates. For the purpose of obtaining the fair value of the property used for the public, fair profitableness is the only true measure of such fair value. Fair profitableness, however, in turn depends on, not the fairness of the individual rate, but the fairness of the general rate level. In such valuation work, it can be assumed that all the individual rates are equally fair, or unfair, to the carrier and that, consequently, the raising or lowering of the average rate level by an uniform percentage, is the remedy for unfairness, if such exists. The fairness, or unfairness, of the average rate level is determined by the one consideration that it will, or will not, afford a fair return on the fair value of the property used for the public.

Fair value, therefore, becomes an indispensable factor in the determination of the fairness of the general rate level.

5.—The Relation of Risk to Rate of Return.

Liquid capital, in its choice of a field of investment, cannot be coerced; the direction of its movement is controlled solely by economic conditions. Provided it is merely a risk and not the certainty of loss, capital in seeking investment is not primarily concerned at the pres-

ence of risk, for it is able to protect itself by varying the rate of premium demanded.

No man makes his individual investment on any other basis than the hope that his particular investment is to be numbered among the successful ones. Many prove unsuccessful, however, and these failures, becoming more or less known, produce fear, great or little, according to their ratio to the successes. This fear must be overbalanced by the offer of a premium over safe investment rate. This premium is in the form of the known high profit made by the successes. If the premium does not appear to be high enough, the investor goes elsewhere.

In order that capital shall continue to enter a given industry, it is necessary that the return, risk considered, on capital invested in the industry as a whole must equal the return from investment in other industries. This is a matter of fundamental economic law and is not permanently susceptible of modification by Court or Commission.

Suppose that, from the beginning, a single capitalist had furnished all the capital for each and every individual enterprise in a given industry; one in ten of his investments, measured by the amount of capital invested, proved failures, and the capital invested therein has been destroyed; the remainder are still live enterprises producing various degrees of return (or even loss) on the investment. In order that the capitalist shall be as well off to-day as if he had simply deposited his capital in a savings bank at 3% interest, he must have received from the successful industries a return on his total investment equal to 3% per year plus the amount of capital destroyed to date. Stated differently, he must have received each year on the total capital invested in still living enterprises a rate equal to the rate of return on safe investment plus the annual rate of destruction of capital in the industry.

If regulation holds the rate below this economic level, it may be possible for a time to entice unwary investors into the industry; sooner or later, however, its rate of development will be retarded if not destroyed. To hasten development, rates must be above normal. If the risks of the industry vary in different sections of the country, the rate of return will likewise have to vary, or those sections where the risk is greatest will suffer.

6.—How Can the Fair Rate of Return on the Fair Value of Property be Ascertained?

Most railroad property is owned by corporations the securities of which are more or less actively traded in on the stock exchanges of the country; consequently, the current prices can be ascertained with little difficulty. Also, the records of the corporation show the amount of such securities outstanding from time to time in the hands of the public. The current prices multiplied by the amount outstanding give the public's opinion from time to time as to the market value of such property. The net earnings of the property from time to time are also ascertainable from the records of the corporation. Both the net earnings and the public's opinion as to market value are constantly fluctuating, but if we take average figures for each over a considerable period of time and divide the average net earnings (profitableness) by the average market value, the result will closely approximate the rate of return which, risk considered, the public demands on railroad securities before it will invest in them.

7.—General Observations on the Valuation of Properties Subject to Competition.

The presence of competitors greatly complicates the problem of valuation, for the reason that there are, in such cases, many intangible factors which affect profitableness and which cannot properly be overlooked. For example, suppose all the railroads in a certain self-contained district were owned and operated as a single property. The methods suggested for the valuation of a monopoly could readily be applied. Suppose, however, this single railway system is split up into several more or less competing systems under different ownership and operation. It still holds true that profitableness under fair rates is the measure of fair value in each individual case; but how shall we compute this profitableness?

In the case of a railroad forming a part of a group in which the lines are in competition, one with the other, some of the elements which affect profitableness are:

Affecting Number of Units of Output.—

Length of road;

Density and character of adjacent population;

Number and character of industries adjacent;
Traffic production or absorption capacity of connections;
Traffic capacity of the railroad—number of main tracks, character and size of terminals, number, character, and size of available equipment, etc.

Affecting Unit Price of Output.—

Rate regulation;
Length of haul;
Character of product transported;
Intensity of competition;
Percentage of through rate secured, etc.

Affecting Unit Cost of Output.—

Maximum grades and curvature;
Length of haul;
Character of product;
Character of equipment;
Adequacy of facilities;
Density of traffic;
Character of agreement affecting interchange of traffic, etc.;
Regulations affecting quality of service, etc.

Miscellaneous.—It is usually customary to enumerate among the things which make profitableness possible:

The presence of the physical property;
The possession of a franchise permitting of the operation of such property;
The fact that the period has already been passed through during which occurs the development of business to the point where its volume is sufficient for fair profitableness (this is usually called "going concern value");
The fact that the property has been constructed so as to permit of unity of use and connected operation instead of being constructed so that it cannot be so used or operated;
The fact that the management has created friends who become the customers and thus the source of revenue of the property.

Remembering that value is measured by profitableness, the value to an individual railroad of the possession by such railroad of the various elements of profitableness (and, therefore, of value) is measured, in the case of each such element, by the effect produced by the presence of such element on the total profitableness of the railroad. These various elements can affect the profitableness of the railroad as a whole only by increasing earnings, or by reducing expenses, or both. In order, therefore, to compute value in this manner, it is necessary to compute separately for each element of value of the property, its effect on earnings and expenses (and, consequently, on profitableness).

To attempt thus to compute value by separately computing what part of the total value inheres in each of the various elements of value previously enumerated (and the many others which are suggested by such enumeration) is evidently to become involved in a hopeless mass of detail from which useful results cannot emerge. It would be useless to attempt to compute the value of a few of these elements unless the value of the sum of the remainder is known.

The composite effect of all the various elements of value possessed by the individual railroad property is accurately weighed and registered in the profitableness resulting from operation under the existing conditions. This, however, merely gives market value and not necessarily fair value (which would be measured by profitableness, present and prospective, under fair commodity rates). How then shall we proceed to estimate fair value?

One suggestion advanced is that, in such case, fair value equals cost plus "other values and elements of value". The only true measure of value is profitableness. The profitableness of an individual railroad, subject to competition, bears no necessary relation, and can be made to bear no fixed relation, to cost. Therefore, to state that the value of such a property is equal to its "cost" plus "other values and elements of value" does not in any way advance the process of computing such value. It is merely equivalent to stating that the weight of a steer equals 500 lb. plus certain other weights yet to be ascertained.

8.—The Relation of Depreciation and of Appreciation to Fair Value.

Depreciation is a lessening in value as compared with value under the basic condition (called "new"); depreciation is brought about by any cause which reduces profitableness, present or prospective. Appre-

ciation is the opposite of depreciation; it may be regarded as negative depreciation.

Solidification of roadbed, grassing of slopes, etc., adjustment of working parts of machinery, etc., reduce future maintenance costs; development of traffic increases net earnings; both, therefore, add to the profitableness and thus produce appreciation in value.

Physical deterioration, obsolescence, and inadequacy affect profitableness by, at some future time, increasing gross expenses as compared with the gross expenses necessary in the absence of such conditions and so produce depreciation in value. The usual remedial process is the replacement of certain portions of the original property at the cost of the expenditure involved in restoring original conditions. This expenditure is usually termed "maintenance cost". Under certain conditions, this cost matures at regular intervals in amounts which are not embarrassing. Under other conditions, the increased cost accrues over long periods of time until, at maturity, it assumes proportions which may produce disastrous results in three ways, namely:

The treasury of the enterprise may suddenly be called on for large sums which it may not be in a position readily to supply and which, therefore, might involve the enterprise in financial difficulties.

This gradual accrual may not become apparent in the ordinary annual reports of the corporation to its stockholders. These stockholders, as well as intending stockholders, basing their estimates of value of the property on the reported past net earnings, form an entirely erroneous conclusion, because they do not realize that the future earning power will be much diminished when it becomes necessary to make the renewals required on account of the heavy accruals of deterioration.

If the commodity rates are under regulation and subject to frequent adjustment, the fluctuations of earning power brought about by gradual accrual and sudden maturity of deterioration, cause an unequal distribution of the production cost among early and late purchasers of the product.

For these reasons it has become customary, from time to time, to include in operating expenses an amount for "depreciation", more or less approximating the annual accrual. These amounts of "depreciation" are included in current operating expenses and a book credit is set up, against which credit the actual expenditures to remove matured

deterioration are charged as they occur. Sometimes, in addition to the mere bookkeeping, an actual fund is created, segregating the amount charged to "depreciation" from the other gross earnings of the corporations, thus establishing a more complete protection.

Various methods have been proposed for estimating the amount of accrual of deterioration, which approximate more or less closely to the actual accrual. The object in each case is the same, namely, to distribute maintenance cost more uniformly than if only expenditures for replacement of matured deterioration were charged thereto.

It is evident that the item which is a final charge to maintenance expense is the cost of removal of deterioration (or "depreciation") and not the deterioration itself, which latter is a condition and not an expense.

In the absence of removal, the present worth of the future cost of such removal is evidently the measure of the depreciation in value as compared with the value if the removal had already been completed, all other conditions remaining unaltered. It is to be observed that, in properties made up of great numbers of articles, as the ties in a railroad track, there is a constant tendency of the amount of deterioration —of the loss of useful life—to reach 50% and to remain constant at that level. Such deterioration in the property as a whole never is removed—the property is designed so as to be really in normal use-condition only when this status is reached—and there is, consequently, no future cost of removal to consider and of which to compute the present worth.

In the case of appreciation, removal thereof is neither necessary nor desirable; nevertheless, the production of the appreciation, whether voluntarily or involuntarily, has usually been accompanied, in some form, by cost; this cost is either chargeable to maintenance or operating expenses, or it must be regarded as a capital expense incurred in the building up of the property to its present condition. As a matter of fact, the cost of that appreciation which takes the form of solidification of roadbed and ballast, grassing of right of way, adjustment of working parts of machinery, etc., ordinarily enters into operating expenses in railroad accounting and can be segregated only with difficulty, if at all, from the other items of operating expense, making it difficult to treat as a capital expense item.

Under the "competitor" method suggested for finding the value of a property, the first step is to estimate that fair rate level which, within that reasonable time which capital would be willing to wait, will produce net earnings for the competitor equal to a fair return on the competitor's cost to date. In order to determine this rate level, it is necessary to estimate, over a period of years, the gross and net earnings of the competitor. Whatever method of computing "depreciation" is in use on the property under valuation, should be applied to the estimation of the maintenance cost of the competitor during the period of years used for the determination of the fair rate level. Having in this manner determined the fair rate level, the next step is to estimate the profitableness, present and prospective, under the application of such rate level, of the property under valuation; this involves the estimating of the gross and net earnings, under such rate level, of the property under valuation. Here, again, the same rules with regard to the computation of "depreciation" should be applied to the estimation of operating expenses during such period, bearing in mind, also, the physical condition of the property and the probable desirability of increasing or decreasing the normal "depreciation" rate on account of such physical condition.

Attention is called to the fact that it is not directly the physical condition of the property which affects its value but, on the contrary, it is the effect of such physical condition on present and future operating expenses which influences value. It is probable that, under the valuation method proposed, it does not seriously modify the result whichever of the various "depreciation" rules is adopted in the computation, provided the rule is sufficiently accurate to produce, in the long run, a close approximation to actual maintenance expenditures and not, on the contrary, gradually pile up a considerable surplus or deficit.

9.—The Relation of Abandoned Property to Fair Value.

As it is the value of the property at present used for corporate purposes which is to be obtained, and as the value of such property is entirely independent of the amount of the property abandoned, it is evident that such abandoned property is to be ignored in arriving at fair value.

However, if the abandonment resulted from unavoidable causes, such as normal obsolescence, etc., the relation between the cost of the

property abandoned and the cost of the remaining property still in use is of interest, because of the light it throws on what is a fair rate of return in the industry in question. Fair rate of return is made up of two factors, rate of return in the absence of risk, and insurance premium to cover risk. The relation between abandoned property and the total property furnishes an indication of the degree of risk. Also, the subject is of considerable interest in connection with the establishment of rules governing the application of depreciation.

It may be well here to remark that it is only the value of the remaining useful life which is abandoned and that, in practice, this is generally reduced to a minimum by neglecting repairs for some time previous to abandonment and while still keeping the property in service.

10.—The Relation of Aids, Gifts, Grants, and Donations to Fair Value.

In practically all cases, the object of gifts, grants, etc., in whatever form these have been made, has been to induce capital to undertake an adventure in which the risk was otherwise prohibitively high. This additional inducement was really in the nature of a capitalized payment of an extraordinary insurance premium.

If, in valuation for the purpose of determining fair return, the value of the gifts should be excluded, then the effect of the gift is merely to put the giver in the position of being a silent partner in the business without in any way reducing the risk of the other partners; yet this reduction of risk was the primary object of the gift.

For example: The estimated cost of a project is $10 000 000, and it is further estimated that, under the commodity rates which must be used, the profitableness of the enterprise will not be more than $400 000, and may be less. As it is impossible to interest capital in an uncertainty with a maximum return of only 4%, certain outside parties, expecting great indirect benefit from the development of the industry, donate $5 000 000 to the corporation formed therefor. This means that the stockholders will be obliged to contribute only $5 000 000 instead of $10 000 000, while the plant which they will own will have cost $10 000 000 to produce. On the assumption that they will be permitted to have the entire return from the $10 000 000 investment, the probable return on the stockholders' investment of $5 000 000, therefore, become 8 per cent. If, on the contrary, the industry is to be subject to regulation and, because of the $5 000 000 donation, the rates

are to be cut to the point where only $200 000 is allowed to be earned, it is evident that the stockholders have not been benefited in any way by the donation, for their risk has not been reduced in intensity and they are still limited to a prospective maximum of 4% on their investment.

Under the method of valuation suggested, it is evidently of no concern to the prospective competitor whether the existing enterprise bought, begged, or stole its now owned property. It is the cost of developing and operating the competitor's plant which must determine the rate necessary to enable it to compete. Fair value of the property under valuation will be determined by its profitableness at this rate.

11.—The Relation of Property Owned but not Used by the Owner in the Public Service, to Fair Value.

It is self-evident that such property should not be included in the value on which fair return is to be allowed. It is to be regarded merely as an investment in which the public has no interest, and any returns from which should be excluded from operating revenue.

It is frequently necessary, however, especially in rapidly growing communities, to look well ahead of the present needs. This sometimes raises the question whether the property is or is not properly to be included in the valuation.

12.—The Relation of Property Used but not Owned to Fair Value.

The value of such property to the user is reflected in the net earnings directly and indirectly accruing from such use. Ordinarily, the gross earnings from such a property are included with the gross earnings from other properties in such manner as to make segregation practically impossible. Under such circumstances, the simplest method of handling for valuation purposes is to include in operating expenses the rental of such property, excluding the item entirely from the reproduction inventory; in some special cases it may be necessary to consider whether or not the terms of the rental are fair, or whether they represent unwise and improvident management, the results from which the public does not subscribe to.

Where it is possible to segregate from other sources of revenue the earnings accruing from the rental of a property not owned but used in the public service, and where the property bears a sufficiently impor-

tant relation to the remainder of the property under valuation, then it can be valued in the same manner as if owned and used. In this case, however, the rental must be excluded from consideration as earnings.

13.—The Relation of Property Owned and Used Jointly to Fair Value.

Recalling that the only true measure of value is profitableness, it is evident that the value to each of the users of a property utilized jointly is measured by the profitableness (gross earnings less gross expenses) accruing to the user from such property, regardless of the terms of ownership. Ordinarily, as the gross earnings accruing directly and indirectly from the use of such a property cannot be segregated from those accruing from other sources, the simplest method of valuation is to include in the gross expenses of each owner its proportion of the expenditures incurred on account of such use. The reproduction cost of the property is pro-rated to the different owners on this or other equitable percentage basis. Where the jointly used property is separately operated and the terms of ownership include the basis of distribution of profit or loss from such operation, the valuation of the jointly used property can be made independently and pro-rated to the users on the basis of the contract basis of distribution of profitableness.

14.—When Does "Construction" End and "Operation" Begin?

In the determination of the competitor's investment to date and the commodity price necessary to the securing of a fair return thereon, a clear understanding of the physical and accounting procedures involved is necessary.

The construction and development of a railroad property to the point where it is a paying going concern involve the following steps: First, the inception of the project and such preliminary promotion and financing as will permit of general investigation and incorporation of the project; second, the securing of sufficient capital to enable the preliminary and location surveys to be made and construction to be commenced. In most cases the purchase of right of way must slightly precede, locally, the commencement of construction. With the continuance of construction, additional capital must be secured. Finally, the point is reached where trains hauling passengers and freight are run, thus enabling capital to be secured from the sale of transportation as well as from the issuance of stocks and bonds.

With the beginning of construction, the processes of solidification, seasoning, and deterioration commence, and these processes increase in rapidity of action with the commencement of train service, thus necessitating expenditures for maintenance of the property already created; embankments settle and are partly destroyed through the action of the elements; grasses, etc., begin to cover the exposed surfaces; decay and wear begin to produce the necessity for later replacement. Meanwhile, construction continues; expenditures for operation and maintenance increase, and the density of train service and of traffic increases.

A time finally comes when it is arbitrarily stated that construction has been completed and that operation of the property has commenced. The failure to recognize that this dividing line is an absolutely arbitrary one, adopted merely for the purpose of facilitating organization matters, including the application of certain accounting rules, and having no relation whatever either to the cost or to the value of the property, leads to confusion in connection with valuation. The continuance of construction which, on ordinary railroads, never actually comes to a state of completion, is now called "additions and betterments"; in some cases it is called "maintenance". In ordinary accounting rules, there is considered to be a great distinction between construction, additions and betterments, operation and maintenance, and "depreciation". For valuation purposes much confusion is avoided if all these terms are entirely disregarded and if, in their stead, the receipts and expenditures necessary to the full development of the property are directly considered without first subjecting them to such misleading classification. In this way the arbitrarily established dividing line between receipts, depending on their sources, and between costs, depending on the instruments used in producing the same, is made unnecessary.

15.—As of What Date Should the Valuation be Made, and What Are "Normal Prices" as of the Date of Valuation?

For practical use, the fair value of a property which must be used is that as of the date or period of use. Thus, for use in condemnation, it must be the fair value of the property as of the date of condemnation.

For use in the regulation of rates, it is necessary to bear in mind that it is undesirable to make frequent modification of rates, but that, on the contrary, they should be determined so as to be fair, on the

average, over a considerable period of time. This necessitates the drawing of conclusions as to the probable average conditions as to prices, etc., which will exist during such future period. The unit prices most satisfactory to be used for valuation for such purpose are those which, it is considered, will probably be a fair average of the prices over such future period. The only method of estimating probable future conditions is through the study of the past conditions; therefore, the ascertainment of average unit prices year by year over a considerable period of the past is necessary. In determining the average unit price for a given period of the past, actual sales furnish the basis. Inasmuch as these sales vary greatly in quantity, it is necessary that the average used be a weighted average and not merely an arithmetical average.

Normal price as of a given date may be defined as being the market price as of that date, assuming business conditions to be normal and not under the influence of undue optimism or pessimism as to the future.

16.—How can Valuation be Kept up to Date for Rate Regulation Purposes?

It must be borne in mind that valuation is not and cannot be made an exact science; neither is there any practical necessity that it should be so. The true measure of the value of a property, as has already been stated several times, is its profitableness, present and prospective, actual and anticipated. Evidently, prospective and anticipated profitableness in themselves can be estimated only approximately, and yet, in nearly all commercial transactions, it is the future prospect rather than the present fact which determines the investor's conclusions as to present value.

Having determined, with such reasonable accuracy as may be, the present value of a piece of property (future prospects being one of the vital considerations), the most important factors which, in the future, will bring about a modification of such estimated value are the appreciation and depreciation in the value of land owned or used, like fluctuation in the prices of labor and material such as have already entered in the construction of the property; the invention of improved methods of construction; additions and betterments necessary to the growth of the business will add quantities to those existing as of the date of valua-

tion; and property abandoned on account of obsolescence, etc., will reduce the valuation quantities. All these changes over a reasonable period of years, together with their effect on the future profitableness of the property, are considered in arriving at the original valuation; further, it is greatly to the interest of the public to avoid frequent fluctuations in commodity prices (freight and passenger rates, etc.). Presumably, therefore, the fair rates established at the time of the valuation (which rates are based, not on the conditions as of the instant of valuation only, but on prospective conditions as well) will be fair for a considerable period of time thereafter. Fluctuations due to additions and betterments and to abandoned property can be taken care of with considerable accuracy by the application of ordinary accounting methods. Variation in the fair value due to fluctuation in unit prices of labor and material can probably only be dealt with either by a new valuation from time to time, or by the keeping of a suitable index number such as that maintained by Bradstreet's, the *London Economist,* and others, but adopting a base arranged to represent the more important kinds of labor and material used in railway construction in their proper proportions, and using the fluctuations of this index for the purpose, from time to time, of applying percentage corrections to the original fair value. Fluctuations in land value could probably only be allowed for through the occasional making of reappraisals of selected properties and through the application of a percentage correction to all the land values.

A percentage correction applied in such manner to the original valuation, although crude, would still probably greatly extend the period of serviceability of the original valuation.

17.—Comparison of the Cost-of-Reproduction Method with the Prospective Competitor Method of Valuation.

1.—Land Values.—The Competitor Method assumes that, in the construction of its property, the competitor will be obliged to purchase land having the same present ordinary values and being subdivided in the same manner as similar lands adjoining the property under valuation as of the date of valuation. This avoids the absurdity, pointed out by Justice Hughes in the Minnesota Rate Case, of assuming that the railroad producing much of the value of the land is non-existent, but that the land retains its full value. Presumably, it might be

claimed that the competitor, in entering a town, would select a location for its line and for its terminals which would cost less per acre than the lands adjoining the property under valuation. The answer is that it is fair to presume that the original railroad located its line in such manner as to make the cost of land plus the cost of grading and other improvements a minimum, considering the value of the location with reference to convenience of access for traffic. The adoption of a different location would mean different grading quantities and also the estimating in terms of earning power of the difference of attractiveness of the two locations. This would involve such excessive complications as to make the problem practically unsolvable. The fair presumption is that the original line secured the best available location, cost considered, and that any other location would cost more or would be worth at least proportionally less. In practical operation and in results obtained, the two methods are identical, as to land. The competitor method is logical whereas the other is not.

2.—Present or Past Conditions.—By the competitor method, all questions as to whether present or past conditions apply in making the estimate are solved without difficulty. The reproduction theory affords no positive clue as to whether present or past conditions should be considered in individual cases, and there is much present controversy over this point.

3.—Construction Period.—The reproduction theory involves the reproduction of the property, as it is to-day, in one operation; thus, a four-track railroad would be reproduced as such absolutely regardless of the fact that no railroad, either the original occupant of the territory or a later competitor, ever is built in such a manner. The competitor theory calls for a gradual construction and development of the competitor at a rate dependent on rate of growth of traffic in the community and, therefore, is consistent with the invariable facts of such industrial development.

4.—Going Concern Value, and Cost of Development of Earning Power.—The reproduction theory admittedly carries the process of estimating value only to the point where the property has prepared to become a going concern but has not taken that step. This leaves an element of value called by the Courts "going concern value" to be ascertained by some succeeding process. No process departing greatly from clairvoyance has yet been devised. The competitor theory pro-

vides for the estimating of the cost of the development of the traffic on a basis readily applied in practice and governed by the application of simple principles to easily ascertainable facts.

III. A Study of the Virginian Railway.

In 1902, the construction of what is now the Virginian Railway was actively commenced by the late Mr. H. H. Rogers, the primary purpose being the economical transport of bituminous coal in great volume to the seaboard. The work was pushed as rapidly as financial conditions would permit. Covering the number of years that it did, the rapid completion of course was interfered with by various periods of business depression. The property is one of the few railroads in the United States which has been conceived and developed as an independent enterprise, most of the present railroad systems having been developed as separate entities and gradually combined. The statistics of this railroad, consequently, are of unusual interest for the purpose of illuminating the various problems connected with the ascertaining of the value of railroad properties.

Operation was commenced on the Virginian Railway in 1910, eight years after the commencement of construction. Thereafter, expenditures for construction (usually termed additions and betterments) were, in the main, for providing the additional facilities required to handle the rapidly developing business. The records of the company, partly owing to the fact that practically all the financing was done by Mr. Rogers himself, do not disclose the amount of capital invested in the enterprise as interest. It will be assumed, therefore, that sufficient capital for the ensuing 12 months was secured at the beginning of each such period and that 6% compounded annually had to be paid thereon. With this assumption, Table 2 gives the essential financial history of the Virginian Railway from its inception in 1902 to June 30th, 1916, and also shows what this history would have been had the rate level been 8% higher than it actually was.

For purpose of illustration, assume that to-day as compared with the period 1902-16:

(1).—There has been no change in construction methods or in unit prices of labor and material or in land values. Assume, further,

(2).—That the history of the Virginian Railway has been unaffected by the presence of competitors.

(3).—That the construction of the Virginian Railway and its traffic development proceeded at a normal pace.

(4).—That its physical condition was normal, that is, there was no matured deterioration, throughout its history.

(5).—That the rate level throughout the history of the property was fair.

Under such assumptions there would probably be few who would question that, as of the date of valuation, the fair value of such a property is measured by its cost to date—in this case, $69 522 883—as of June 30th, 1916.

Taking up the various assumptions and assuming that 6% is the fair rate of return on the fair value of such a property, the following considerations suggest themselves:

1.—Six per cent. on $69 522 883 is $4 171 373, whereas the net earnings for 1916 are $3 331 957. Evidently, on that date, the property was not earning a fair return on its cost. If the rate level throughout the history of the property had been 8% higher than it actually was, then the cost to date would have been $65 996 540; 6% on this amount is $3 959 792 and the net earnings for 1916 would have been $3 923 187, indicating that, based on these considerations alone, the rate level was about 8% too low throughout the history of the property and that, for this reason, the property to the end of 1916 has cost $3 526 343 more than it would have if the higher rate level had been in effect. Had the higher rates been in effect, the public, during the period, would have paid $2 877 237 more for transportation than it did.

Question A.—Should this increased cost equitably fall on the owners of the railroad through the assumption of the lower figure or should it fall on the public through the assumption of the higher figure as being the fair value of the property as of June 30th, 1916, under the above assumptions?

2.—Assume that the earning power of the property had developed more rapidly, so that, in 1914, the net earnings had been equal to the actual net earnings of 1916. This would have required a corresponding acceleration in the additions and betterments, and the cost to date as of June 30th, 1914, would have been $63 632 623 for a property identical in

every way with the actual property as of June 30th, 1916, which actual property cost $69 522 883, a difference of $5 890 260. On this new basis of development of earning power, the rates were about 5% too low to give a 6% return on the cost to date of the property as of June 30th, 1914, whereas they were 8% too low to give a 6% return on the identical property with the development period two years longer.

Question B.—Should this excess cost be borne by the public through its inclusion in fair value as of a given date, or should it be borne by the owners of the property through its exclusion?

Question C.—If the owners of the property should be penalized for their bad judgment in commencing the enterprise in advance of the normal public need, thereby adding to its cost, should they also be rewarded if they delay the carrying out of the project until the public need becomes abnormally acute, thus reducing the cost of the project? If bad judgment is penalized and good judgment not rewarded, what will become of the industry—collapse, or Government ownership?

Question D.—How shall it be determined whether or not the inception of the enterprise was in advance of the normal public need?

3.—Going back to the first case—that of the actual history of the Virginian Railway—suppose the date of valuation had been taken as of June 30th, 1914, instead of June 30th, 1916. In that case the cost to date of the property would have been $63 149 009; 6% on this amount is $3 788 940, whereas the actual net earnings for that year were only $2 641 048.

If the rate level had been 9% higher than it was, the cost to date would have been $62 884 206; 6% on this amount is $3 773 052 as against net earnings at the new rate level of $3 718 861 indicating that, under such conditions, and not considering the future, the rate level was slightly more than 9% too low as against an 8% deficit if the year 1916 had been adopted as the basis of calculation.

Question E.—In 1910 (the year in which the Virginian Railway was opened for operation), in determining the rate level to be used in the future on this property (on the assumption that it was not subject to competition) should this rate level have been based on the estimate that in 1914 it would give a fair return on the cost to date, or should it have been based on the fair return being reached in 1916, or in some other year? What considerations should determine the year to be selected?

Question F.—The coal traffic of the Virginian Railway is in competition to tide-water with the coal traffic of the Chesapeake and Ohio and of the Norfolk and Western Railways. If it is assumed that the actual rate level (which was 8% too low to give a 6% return on the cost to date of the Virginian Railway, as of June 30th, 1916) was just sufficient to give a 6% return on the cost to date of the Chesapeake and Ohio and of the Norfolk and Western Railways, would this fact have any bearing on the fair value of the Virginian Railway?

Question G.—Since the inception of the Virginian Railway, there has been an increase in unit prices of labor and material used in construction and operation and in the price of land along the railroad right of way. Do these facts have any bearing on the fair value of the Virginian Railway?

Question H.—If it is assumed that, for several years just prior to the date of valuation, the Virginian Railway had either been parsimonious or excessively liberal in the renewal of its rail, ties, and other materials, would this fact (which certainly would affect the operating expenses of the immediately succeeding years) have any bearing on its fair value as of the date of valuation?

It is the study of such considerations as those mentioned in the preceding paragraphs which has led very commonly to the conclusion stated elsewhere, that there is no general relation existing between the cost and the fair value of a property. It is evident that, if cost is to be used in the determination of fair value, it must be used only under such special circumstances as permit the relation between cost and value to be known.

The object of the Prospective Competitor Method of valuation is to ascertain the character of these special conditions and the relation which, under such conditions, exists between cost and value.

Applying to the Virginian Railway the Prospective Competitor Method of valuation, still assuming—for the purpose of illustration and because the data are lacking—that the value of the property is unaffected by the presence of competitors and that, since its inception, there has been no change in unit prices of labor and material, in land values, or in engineering methods:

1.—Under the assumption that the original construction proceeded at a normal pace, it is to be assumed of course that the construction of

TABLE 2.—Virginian Railway Actual Original Cost to Date *vs.* Cost if Rates Had Been 8% Higher.

Year ending June 30th.	Year.	Annual construction cost, exclusive of interest.	Annual Gross Operating Earnings, Exclusive of Interest.		Actual annual operating expenses, exclusive of depreciation.	Annual Net Operating Earnings, Exclusive of Interest and Depreciation.		Net Annual Income or Expenditure—All Sources, Except Interest and Depreciation.		Original Cost to Date.	
			Actual.	With 8% rate increase.		Actual.	With 8% rate increase.	Actual.	With 8% rate increase.	Actual.	With 8% rate increase.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1902	1	$4 885	.....	.....	.....	.....	.....	$4 885	$4 885	$5 178	$5 178
1903	2	124 991	.....	.....	.....	.....	.....	124 991	124 991	137 979	137 979
1904	3	803 553	.....	.....	.....	.....	.....	803 553	803 553	998 023	998 023
1905	4	2 002 747	.....	.....	.....	.....	.....	2 002 747	2 002 747	3 180 816	3 180 816
1906	5	8 068 906	.....	.....	.....	.....	.....	8 068 906	8 068 906	11 924 705	11 924 705
1907	6	13 914 302	.....	.....	.....	.....	.....	13 914 302	13 914 302	27 389 347	27 389 347
1908	7	5 713 959	.....	.....	.....	.....	.....	5 713 959	5 713 959	35 089 504	35 089 504
1909	8	6 487 348	.....	.....	.....	.....	.....	6 487 348	6 487 348	44 071 463	44 071 463
1910	9	1 857 966	$2 063 190	$2 228 245	$1 858 980	$204 210	$369 265	1 653 756	1 488 701	48 468 732	48 293 773
1911	10	5 085 982	3 671 224	3 964 621	2 831 760	839 464	1 132 861	4 246 518	3 953 121	55 878 165	55 381 707
1912	11	1 261 087	4 837 598	5 224 605	3 574 473	1 263 125	1 650 132	Cr. 2 038	Cr. 389 045	59 228 694	58 292 221
1913	12	2 341 028	5 842 584	6 309 990	3 460 099	2 382 485	2 849 891	" 41 457	508 863	62 738 471	61 250 359
1914	13	1 472 432	6 340 079	6 847 285	3 699 031	2 641 048	3 148 254	" 1 168 616	1 675 822	65 264 046	63 149 009
1915	14	1 254 047	5 820 406	6 286 038	3 655 936	2 164 470	2 630 102	" 910 423	1 376 055	68 214 840	65 479 331
1916	15	704 748	7 390 382	7 981 612	4 058 425	3 331 957	3 923 187	" 2 627 214	3 218 444	69 522 883	65 996 540
Totals...	..	$51 097 976	$35 965 463	$38 842 396	$23 138 704	$12 826 759	$15 703 692	$38 271 217	$35 394 284	.....	.....

the competitor to the beginning of operation will duplicate the historic construction to the same period.

2.—It will be assumed that the business handled by the competitor during the second year of its operation equals one-half that of the Virginian Railway as of the date of valuation. This provides one year of minor operation during which the roadbed is solidifying sufficiently for the normal handling of traffic.

3.—For the reason that traffic in the United States doubles in about 16 years, the total length of the development period for the competitor is taken as 17 years. At the end of this period, the physical property and the density of traffic are assumed to be identical with those of the Virginian Railway, as of the date of valuation.

4.—Through experiment, it is found that a 13% rate increase over that actually in effect during the history of the Virginian Railway is necessary during the period of development of the competitor's business, in order that, at the end of the development period, the competitor shall be earning 6% on its original cost to date. A comparison of the essential data of the Virginian Railway with that assumed for the competitor is shown in Table 3. Table 4 is a comparison of the actual financial history of the Virginian Railway with that history as it would have been if the rates had been 8% higher, and also with the financial history of the competitor with a 13% rate increase.

An examination of Table 4 indicates that the Virginian Railway was not earning 6% on its cost to date to the date of valuation, but that an 8% rate increase would have accomplished this. The effect of this 8% rate increase is to reduce the original cost to date from the actual $69 522 883 to $65 996 540. This last amount compares with the original cost to date of $70 882 787 for the competitor (with its 13% rate increase). This difference of about $5 000 000 represents the value accruing to the property because of the abnormally good judgment used in selecting the Virginian Railway for investment. This enterprise was located so that its rate of growth of business caused it to double in 6 years, whereas the normal rate of growth of railroad business in the United States requires about 16 years.

None of the figures used in the preceding study of the railroad included any amount for depreciation, although it is probable that about $5 000 000 of accrued deterioration (loss of useful life of the simple parts of the property) existed as of the date of valuation. If it is to be

TABLE 3.—Essential Data, Virginian Railway History *vs.* Competitor Assumptions.

Year of life of enterprise.	Annual Construction Cost, Exclusive of Interest.		Annual Gross Operating Earnings, Exclusive of Interest		Annual Operating Expenses, Exclusive of Interest.	
	Actual.	Programme for competitor.	Actual.	Programme for competitor (13% rate increase).	Actual.	Programme for competitor.
1	$ 4 885	$ 4 885	.....	.....	.....	.....
2	124 991	124 991	.....	.....	.....	.....
3	803 553	803 553	.....	.....	.....	.....
4	2 002 747	2 002 747	.....	.....	.....	.....
5	8 068 906	8 068 906	.....	.....	.....	.....
6	13 914 302	13 914 302	.....	.....	.....	.....
7	5 713 959	5 713 959	.....	.....	.....	.....
8	6 487 348	6 487 348	.....	.....	.....	.....
9– 1	1 857 966	1 000 000	$ 2 063 190	$ 2 260 000	$ 1 858 980	$ 1 800 000
10– 2	5 085 982	1 000 000	3 671 224	4 181 000	2 831 760	2 800 000
11– 3	1 261 087	750 000	4 837 598	4 463 500	3 574 473	2 900 000
12– 4	2 341 028	750 000	5 842 584	4 746 000	3 460 099	3 000 000
13– 5	1 472 432	750 000	6 340 079	5 028 500	3 699 031	3 100 000
14– 6	1 254 047	750 000	5 820 406	5 311 000	3 655 936	3 180 000
15– 7	704 743	750 000	7 390 382	5 593 500	4 058 425	3 265 000
16– 8	.....	750 000	.....	5 876 000	.....	3 340 000
17– 9	.....	750 000	.....	6 158 500	.....	3 420 000
18–10	.....	750 000	.....	5 441 000	.....	3 500 000
19–11	.....	750 000	.....	6 723 500	.....	3 550 000
20–12	.....	750 000	.....	7 006 000	.....	3 600 000
21–13	.....	750 000	.....	7 288 500	.....	3 680 000
22–14	.....	750 000	.....	7 571 000	.....	3 760 000
23–15	.....	750 000	.....	7 797 000	.....	3 840 000
24–16	.....	750 000	.....	8 023 000	.....	3 900 000
25–17	.....	750 000	.....	8 249 000	.....	4 000 000
26–18	.....	727 285	.....	8 351 131	.....	4 058 425
.....	$51 097 976	$51 097 976	$35 965 463	$111 068 131	$23 138 704	$60 693 425

TABLE 4.

Item.	Original Cost to Date, Development Period 7 Years (1910–1916 Inclusive).		Original cost to date of competitor 17-year development period, 13% rate increase.
	Actual.	With 8% rate increase.	
Construction	$51 097 976	$51 097 976	$51 097 976
Less net from operation	12 826 759	15 703 692	50 474 706
Difference	$38 271 217	$35 394 284	$623 270
Interest	31 251 666	30 602 256	70 259 517
Original cost to date	$69 522 883	$65 996 540	$70 882 787
6% interest on cost	4 171 373	3 959 792	4 252 967
Net earnings last year	3 331 957	3 923 187	4 292 707

assumed that this amount should have been covered by adding a depreciation charge to the maintenance expense and an actual cash reserve established, then it is evident that the original cost to date would have been increased by this $5 000 000. Inasmuch as no such cash reserve has been established, the amount of this reserve would have to be deducted from the original cost to date, in arriving at the value of the property as of the date of valuation. The net result is exactly the same as if depreciation had been entirely ignored—as has been done in the preceding computations. There is nothing in the history of the Virginian Railway to indicate that its physical condition as of the date of valuation was such that maintenance expense in the immediate future would be either abnormally high or abnormally low. Consequently, in so far as depreciation is concerned, there is no need for considering prospective profitableness in connection with the determination of fair value.

SUMMARY.

The Prospective Competitor Method of valuation of property subject to competition is dependent for its successful use on the proper selection of two items, for neither of which the method provides a determination which, in its last analysis, does not depend on the judgment of the user. These two items are: (1) the earning power which the competitor is assumed to have at the commencement of normal operation (say, the second year of operation in the case of a railroad); and (2) the rate of increase of earning power.

Inasmuch as the public does not underwrite the cost of abnormal errors of judgment, among which may be classed the inception of an enterprise where there is not yet a normal public demand for the product, and inasmuch as, consequently, the owner of the property is entitled to reward where exceptionally good judgment has been used, it is evident that these two items must be selected so that they represent normal or average conditions rather than those actually confronting the property under valuation.

With reference to the rate of increase of earning power, the problem is comparatively simple, inasmuch as the average rate of growth of earning power of a given industry in a competitive district is readily ascertainable.

There are several points of view from which the consideration of the earning power to be assumed for the first normal year of operation can

be considered. First, we may consider past history. What was the earning power of the first year of actual enterprises of the character under consideration? The objection to using these data is that the earning power awaiting one enterprise might be satisfactory and yet might be entirely unsatisfactory for a different enterprise requiring a greater initial outlay.

A second point of view is that which considers the normal character of capitalization of enterprises. The early expenditures for construction, etc., must be borne by the owners. After construction has proceeded so far that there is a property having a considerable tangible value, then it is possible for the owner to seek financial assistance through mortgaging his property; especially is this true if a definite earning power has been developed. If, therefore, the initial earning power of the competitor is sufficient to pay operating expenses plus interest on one-half the cost to that date, a definite basis will be established for securing additional capital through a bond issue equal to one-half the cost to such date.

To illustrate: The original cost to the end of the second year of operation of the Virginian Railway's "competitor" was about $56 000 000. Interest at 6% on one-half this sum is $1 680 000. Operating expenses for the second year are estimated to be $2 800 000, the sum of the two items being $4 480 000 as against the $3 671 224 gross earnings actually earned by the Virginian Railway during its second year of operation (at rates apparently much too low).

The history of five other properties shows a second-year gross of from $3 300 to $9 600 per mile of road, that of the Virginian Railway being about $4 600 per mile.

DISCUSSION

Mr. Snow. J. P. Snow,* M. Am. Soc. C. E. (by letter).—This very interesting paper shows clearly that when public regulation is applied to competitive industries a different method of arriving at a proper base for return from rates is needed from that which is proper to use in determining the corresponding base for industries essentially monopolistic in their nature. It also shows the impossibility of drawing a line of demarcation between typical public utilities and ordinary industries which depend on the public for customers. The fact touched on by the author that simple industries tend to be regulated automatically by natural competition, cannot be depended on to control sufficiently many classes of business in this organizing age.

Mr. Byers favors a rate base dependent on earning capacity. The writer fully agrees to this proposition as applied to the relative bases in groups of competitive industries, but prefers the straight reproduction cost as the base for essential monopolies. The substitute plant method, with estimated income and assumed rates, recommended by the author, leads to such wide fields of conjecture that it can hardly be approved.

In the case of a distinct monopoly, even when competition is feasible, like Mr. Byers' instance of the small water supply system described on page 1333, the matter should be decided by the question "what is fair" to all interests concerned, which in this case is 10 cents per 1 000 gal. The proprietor of the former water wagon, whose business is disrupted by the new system, although an interested party, gets no compensation under the present laws.

The well established rule of regulation upheld by the Courts so far as their knowledge goes, is to fix rates based on proper cost of service allowing a fair return on the fair value of the property required in that service. There are many difficult details in fixing upon fair value, and one that has led to much discussion is the so-called "going value" of an enterprise. A better name for this element is "development expense", as intimated by the author, and it is to be hoped that the former term will drop out of the literature on this subject as seems to be the present tendency.

In case of competitive industries, as Mr. Byers states, the cost basis cannot apply in fixing individual rates. The railroads of the United States furnish a splendid and tremendous example for discussion of this statement. As a whole the railroads are competitive, although there are many feeder lines which are, in themselves, strictly monopolistic. In general, these feeders are short lines, but this is not true to a sufficient extent to justify treatment as a "short line" class as seems to be the present vogue. The distinction of having only one

* Boston, Mass.

terminal or junction with other lines of rail or water transportation is a better means of differentiation from competitive lines than the terms "short" and "through" systems. The terms "competitive" and "non-competitive" may suffice for our purpose. Mr. Snow.

It is thought that the rate base for the non-competitive class should be fixed upon individually as previously indicated for essential monopolies. For the competitive class, which includes the great bulk of the mileage of the country, the only proper basis for relative rates, in the writer's opinion, is the normal income of the individual roads as compared with each other. The total return on these roads in the aggregate should be a fair rate on their aggregate valuation as determined by the Interstate Commerce Commission, using the system which it has laid down as fair. This aggregate return should be pro-rated among the various roads according to their normal earning capacity as measured by the rental recently agreed on by the Railroad Administration and the various corporations. These apportionments, capitalized at an agreed-upon rate, will be the rate bases or authorized capital of the said corporations.

The new adjustment of capitalization thus provided for is drastic, but it settles the vexed question of strong and weak roads; it satisfies the aspirations of those who hold that fair value is based on earning capacity so far as competitive roads are concerned; and, in the aggregate, it satisfies those who hold that return should be based on proper cost of plant.

The fact that the final valuation of all the roads is not yet completed and agreed on need not delay the determination of individual apportionments of capital, because the work is far enough advanced so that an aggregate fair value for all roads could be computed with sufficient precision. In fact, the probable error in fixing the proper rate of return to apply to this aggregate valuation will far exceed in its effect any lack of precision in the latter. The valuation fixed on for a given road would not affect the rate base of that road at all.

This method, in the writer's opinion, presents the fairest and most defensible way of placing the railroads on a proper capitalization base, that it is possible to devise.

C. E. Grunsky,* M. Am. Soc. C. E. (by letter).—The writer can see no objection to a comparison with a prospective competitive enterprise, as outlined by the author, when public utility rates are to be fixed, but fails to see therein a generally acceptable solution of the problems which the fixing of rates presents. It is fundamental that the legitimately invested capital—computed at present-day prices—should be protected and ultimately returned to the owner (unless the public utility has perpetual life), and that the owner of the utility is Mr. Grunsky

* San Francisco, Cal.

Mr. Grunsky.

entitled to compensation, in addition to interest on the investment, for rendering a valuable service to the public. He has created wealth for the community at large throughout the territory which his public utility serves and is entitled to a share thereof. His reward should appear in earnings somewhat greater than operating expenses and interest on the investment. Generally, the amount of such reward or compensation can be brought into a satisfactory relation to the volume of business, that is, to gross receipts, and after interest on investment and operating expenses have been ascertained, the "going value" can readily be computed from the surplus of gross receipts in excess of these items.* This is in line with the author's statement "that, contrary to the general view, it is practically much more convenient first to ascertain fair rates and then therefrom to ascertain fair value, than to proceed in the opposite direction." He would be quite right in this statement if he had restricted the same to that portion of the rates or earnings which is to create that indefinite something generally referred to as "going value" (to be distinguished from "going concern value", which usually is held to mean the exchange or market value of the business in its entirety).

The author falls into the usual error of the Courts when he says: "In case of rate regulation, the owner is entitled to such rates as will enable him to earn a fair return on the fair value of his property." He overlooks what seems axiomatic, that value results from the fair rates and that what the Courts should insist on is the protection of the legitimate investment plus adequate compensation for having brought the service of the public utility within the reach of a community. The writer in extending these comments on valuation for rate-fixing purposes will make use of his discussion of a recent paper on "Rational Valuation", by Mr. James Rowland Bibbins, prepared for the Spring Meeting of the American Society of Mechanical Engineers, at St. Louis, Mo. (May, 1920).

Mr. Bibbins is right when he says, in the paper referred to, that, in the matter of valuation procedure, the Engineering Profession should establish the guide posts of progressive thought, courageously, fairly, and without undue dependence upon legal precedents, giving due weight to decisions of the Courts and commissions, but only in so far as these are sound in the light of true engineering economics." The difficulty which has frequently resulted from too strict adherence by valuation engineers to the opinion of the Court in the famous Smyth *vs.* Ames case, that value must be made the basis of the calculation, the starting point, when public utility rates are in question, is well known. How earnest has been their effort to reconcile value and the rate base, and how frequently have they been driven to cover in attempting to

* "Public Utility Rate Fixing", by C. E. Grunsky, p. 55.

demonstrate value when they should have been looking for something else that would be a better basis for a rate calculation. They have allowed themselves to put into the word "value" new meanings, making it stand for something which is not value at all. Then, there is that other illustration, the decision of the United States Supreme Court in the Knoxville case, which apparently closes the door against past history with the words: Mr. Grunsky.

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own."

Referring to the need of being conversant with the historical facts in controversies between corporations and the public, Mr. Bibbins, in the same paper, says:

"It is no exaggeration to say that much of the controversy over valuations could have been avoided entirely had it been possible for the contending parties to view the complete facts concerning which there could be little or no argument."

He is right again, and it is to be hoped that the better access to facts which may be expected in the future—after some years of operation under control—will go far toward establishing that mutual confidence which is essential as a basis for satisfactory service, and will open the door to full historical review, despite such views as have been expressed in the Knoxville and other cases.

The simplest possible valuation problem may be selected as best adapted to illustrate fundamental principles. It will not be necessary to give more than passing note to the more involved problems which require analysis of franchise obligations and restrictions or which are concerned with enterprises subject to a bonded debt. Neither need valuation of any property be considered the rates of which are not subject to regulation, because it is only when the earnings of a property are subject to regulation that the valuation expert has to deal with the problems that have been rendered obscure by the requirement that ascertainment of value shall precede and be made the basis of the determination of net earnings.

The Courts are required to prevent the taking of property without adequate compensation. Whenever public utility rates are established so low that present and prospective earnings will not yield a fair return on the legitimately invested capital, the owner of the utility properties will find that either he is being deprived of property without his consent or that the enterprise was ill-advised and cannot have a value fully equal to the legitimate investment. The Courts when inquiring into the sufficiency of rates are concerned, therefore, with both the legitimate investment and with value.

Mr. Grunsky.

By legitimate investment is meant the reasonable present-day investment which would be required to reproduce the properties in question. It may refer to these properties either in a new condition, or in their present-day or depreciated condition. For rate-base purposes it may suffice to know only the former. For valuation purposes in the proper sense of the term the latter must be ascertained. In the case of every going concern, the investment will include such items as the cost of establishing the business, it being always assumed that the included items can be classed as legitimate.

For the purpose of illustrating fundamental principles, consideration may now be given to the simple case of a public utility enterprise subject to rate regulation, which has been financed by its owners and is not burdened with debt. If such a property is owned by a corporation, as will usually be the case, the stockholders will have been assessed to provide the capital for the investment in the public utility properties. On this capital the owners will expect and should get a fair interest return. The earnings, present and prospective, must be adequate to meet all operating expenses, including a fair allowance for the replacement of the perishable parts of the plant whether consumed rapidly or slowly, and, in addition, they must provide for an adequate interest return on the investment. The earnings must also provide for the gradual amortization of so much of the original investment as will leave a balance on which interest should be earned in perpetuity. This is on the assumption that the public utility in question will have such a long probable life that it may be regarded as perpetual. If, however, the term can be fixed either accurately or approximately in which the utility or any other business will run itself out, the earnings must include a further allowance for the creation of a surplus which, in the probable life of the business, will return to its owner the entire amount of his investment, except, of course, any residual values which may be recoverable when the business is closed out. All this is fundamental and simple; but in the practical application there may be great variation from this or any other clearly defined method of procedure.

Ordinarily, for example, there will be losses in the early years of operation. Provision may have been made to amortize these within a reasonable time, or they may have been regarded as a part of the cost of establishing the business and be treated exactly like other capital invested in the business. When, after a time, rates are to be regulated this cannot be done without historical inquiry, and if such inquiry is barren of results, arbitrary action, such as circumstances seem to warrant, may have to be taken.

Or, again, when public utilities are under investigation it may well be asked: What should be the rate of return? Should this rate be the same at different times and places and for different utilities?

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And what treatment shall be accorded to that public utility which can show a large output of service as the result of a trifling investment of capital? What intangible values shall the earnings be allowed to create? These questions at once suggest that full justice will not be done when the owner of a public utility is allowed a bare interest return on the legitimate investment. The public utility has contributed to the prosperity of the community. Its owner is entitled to share in this prosperity. A bare interest return on his investment would be inadequate. Not only this, but he is entitled to compensation for the hazards which he has assumed, for his ability to command funds, for his superior business skill, and for management. In ordinary private business affairs compensation for the service such as he is rendering, apart from salaries, would most likely be based on the volume of business. Why not also in the case of the public utility? If this principle of compensation is accepted, there need be no further effort to evaluate "going value" as a component part of a rate base. The aggregate value of all intangible elements in excess of the cost of acquiring them, would result at once from a capitalization of all net profit, including the proposed commission on the volume of business in excess of interest on the investment. Going value will then appear as a result of the rates and not as a part of the rate base. By such a common sense inversion of present-day procedure, the difficulty will be overcome of fixing the fair rate of return in the case of the express company and other public utility corporations which may do a tremendous business on a trifling investment. The trouble in these cases heretofore has been caused by the endeavor to find something intangible to evaluate, in order that the value of physical properties, or the apparent investment, as the case might be, could be swelled to such magnitude that it might with some propriety serve as the rate base.

No satisfactory basis, however, has yet been found for evaluating all the intangible elements. In some cases, to be sure, it can be shown that it has cost something to establish the business. Such cost is usually a legitimate investment and, therefore, properly incorporated in the rate base; but it is not a measure of going value or of any other intangible element. If it was, a premium would be placed on bad management or on the least advantageous venture, which is a repugnant proposition. Because in all the years in which valuation and rate-fixing problems have been receiving the best thought of engineers and economists, no one has suggested any satisfactory method, standard or norm, for measuring the value of intangible elements other than with the aid of capitalized net earnings, should not the engineer be the first to admit the absurdity of "trying to lift one's self over the fence by pulling on one's boot straps", and, having made this admission, should he not contend for a fair excess of earnings over operating expenses, including interest on the investment,

Mr. Grunsky.

thereby creating such intangible values as will fit the circumstances of each case.

An express business may serve again to illustrate this point. Suppose that the legitimate investment in physical properties on which there should be a return, is $1 000 000, that the cost of doing business, not including interest on the investment, is $3 000 000, and that the outlook for a continuation of the business is such that amortization of capital need not be taken into account. Ordinarily, the value of the business as a rating base would be sought by adding some kind of an estimate of going value to the $1 000 000 of legitimate investment; but without considering net earnings—which the rates are to produce—there is no way of getting at going value. Why not, therefore, proceed as follows: The express company is reasonably entitled to a 10% commission on the volume of the business transacted, besides fair interest on its investment in physical properties. Interest on this investment at 8% would be $80 000. This amount added to the cost of operation and to the commission which, at 10% of the gross receipts, would be $342 000, will make $3 342 000. The aggregate excess of receipts over operating expenses should then be $342 000, made up of $80 000 interest on the investment and $262 000 as the earnings attributable to going value. This amount capitalized at 8% will make the going value, in this case, $3 275 000.

The work of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities,* of the Society went far to simplify the problems which are presented to the valuation expert, but the Committee lacked the courage to place itself in opposition to the decisions of the U. S. Supreme Court. It failed to make the point that value cannot be the rate base when public utility rates are to be fixed, because value results from earnings present and prospective and earnings result from the application of rates. It also failed to recognize that instead of evaluating going value as a constituent element of the rate base, earnings should be allowed, which, in the opinion of those who have themselves had experience or who are conversant with the business experience of others, will, in due time, create a fair going value. Naturally, in reaching a conclusion on this subject, the expert will compare the going concern with a new business of similar character and will endeavor to ascertain what early losses and other sacrifices may reasonably be assumed to be or to have been necessary to make the business a going concern and, by his findings in this respect, he may shape his conclusion as to what going value should be created, in order that the owner may ultimately recover the full measure of his legitimate investment and that he may be fairly compensated. As

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1311.

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stated in another connection, however, the early losses and deficiency of earnings in the early years are not necessarily the measure of any intangible value.

In this matter, the Committee correctly notes that there may be a legitimate cost of establishing a business and is right, too, when it says:

"The Committee aims to clarify the subject by assigning all costs * * * to the appropriate items of the reproduction schedule, and by classing the costs and losses of the development period as development expense and confining going value strictly to its intangible aspect, depending on a study of the income account which reflects the advantages accruing from enlarged business, social growth, and other factors of improvement and progress."

As previously noted, every public utility—necessary utilities being assumed—should be allowed to share in the general prosperity which it helps to promote. No hard and fast rule, however, can be laid down for determining what the utility's share should be. Take, for example, the Water Company which owns broad acres for reservoir and watershed purposes, and apply the principle laid down in the Consolidated Gas Company case in which the Court says:

"And we concur with the Court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property which legally enters into consideration of the question of rates has increased in value since it was acquired, the Company is entitled to the benefit of such increase. This is at any rate the general rule. We do not say that there may not possibly be an exception to it where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such value unjust to the public."

If this view of the Court is accepted, as it is by those who consider rate base and present value synonymous, the question is where to stop. It might, as the Court foresees, be quite as improper, in certain cases, to allow to the owner of a public utility a return on enormously increased values of real estate, as it would be, by fixing inadequate rates, to prevent the owner of a utility which has no appreciating items of property, from enjoying any of the unearned increment which he has helped to create.

It should be noted in this connection that appreciation may result from various causes. The increasing population of the country with the resulting development of resources brings land progressively from a lower to a higher use. With the demand for each higher use the value of the land increases. Certain types of property improve and become more valuable with age. This is true of earth embankments. Or, again, property may become more valuable by reason of the shrinking value of the money unit, which is at present painfully apparent in the long continued rise of prices. How then shall a

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conclusion be reached as to the legitimate investment? What is the sacrifice of the owner of the utility which is to be protected? Surely not an evaluation thereof on his actual money investment, as, for example, in the first years of this century when the dollar would buy two and one-half times as much as it will buy to-day? Must present-day prices then be considered? And, if so, what are the present-day prices? Shall those prevailing at some instant of time be applied? Preferably not; perhaps it will be best to apply average prices during a period recently passed, in which it may reasonably be assumed that the property in question could have been reproduced. Under the ordinary procedure of rate-fixing, however, will it then be possible to escape from the necessity of frequent re-valuation and re-fixing of rates if this principle is to be applied? Or, putting the problem in a somewhat different way, does not the fixing of rates for a definite or indefinite period of time involve more or less speculation, based on the gain or loss which may result from a fall or rise in the value of the money unit?

Organized labor, for example, is fairly successful in securing higher wages when the cost of living increases, when the value of the dollar goes down. Would it not be possible without frequent re-valuation to find a method under which rates would be changed automatically whenever there is a material change in the value of the dollar? Value is here, of course, used in the sense of desirability in exchange. Money is valuable only because it can be exchanged for things which man has come to regard as necessary or desirable. It is convenient as a medium of exchange; but it is not a desirable standard on which to base future payments, though serving well as a standard of value in transactions which do not involve deferred payments.

By common consent, valuations are made in terms of money; but, as stated, money is not a satisfactory standard of value when the time element involved in a transaction is material. The valuation of five years ago is not applicable to-day, not only because there have been relative changes of value as well of labor and of commodities, but, also, for the more cogent reason that the purchasing power of the dollar when measured by the things which are desirable and of which many have entered into plant acquisition and construction, has materially decreased. It is, fundamentally, just as important to use a dependable standard or norm of value as it is to apply correct principles of valuation and rate-fixing. After the experience of the last five years during which, under the influence of currency inflation and other causes, commodity prices and wages have been vaulting to unprecedented heights, can any one still claim that the money unit—the dollar—is an all-round satisfactory standard of value? The best that can be said for money in this connection is that, although it serves as a

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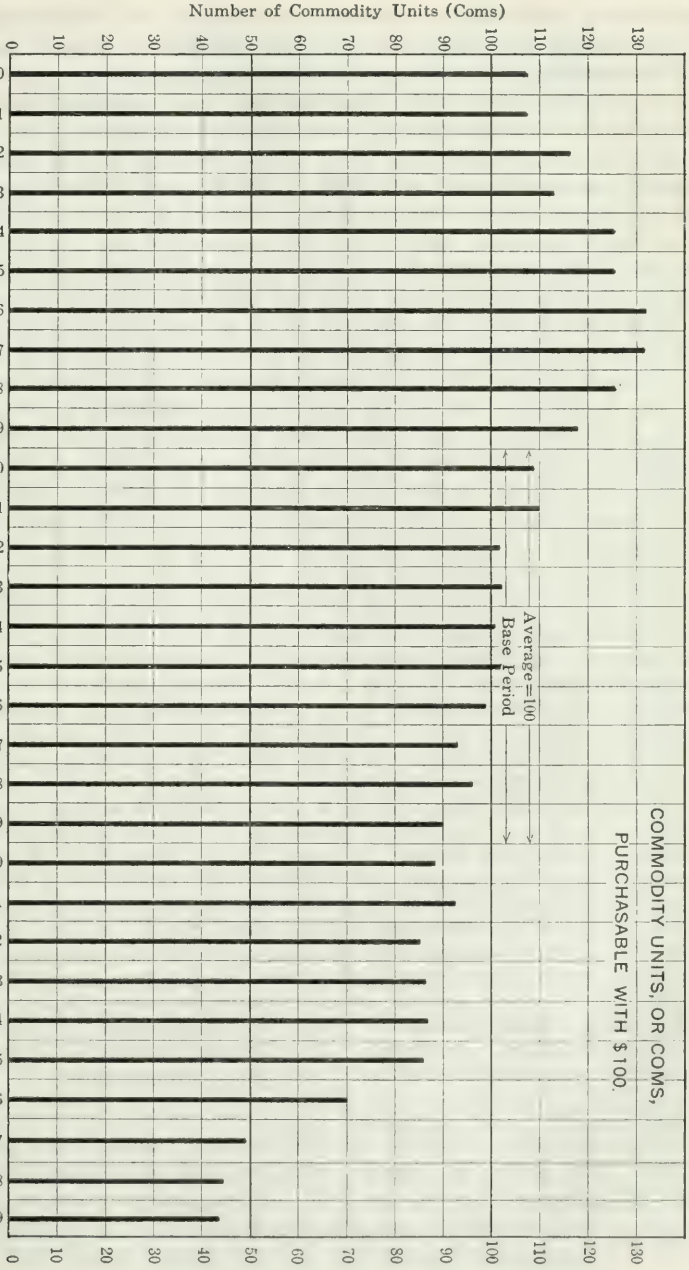

FIG. 1.

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convenient and necessary medium of exchange, its serviceability as a sole standard of value is questionable, and its use as such should be discouraged whenever the time element is material.

To illustrate this fact a diagram, Fig. 1, is submitted herewith to show the variable quantity of commodities of all kinds, in the proportion of their annual consumption in the United States, which $100 would buy in each year since 1890. This diagram, originally prepared by the writer for the Commonwealth Club of California, is based on the index numbers which have been computed by the Bureau of Statistics of the United States Department of Labor. The wholesale prices of from 250 to more than 290 different commodities properly weighted are used by the Department in making the calculation.

As a base period the ten years, 1900 to 1909, have been selected. This was a period of fairly normal conditions. It is more suitable as a base period than the preceding ten years (sometimes used by economists), for the reason that these earlier years cover a period of business depression in which the dollar had more than normal purchasing power. The writer in the paper read by him in February, 1920, before the Commonwealth Club,* suggested that a definite commodity unit be established to supplement money and said in this connection:

"In view of the defects of money as a standard, having value which changes in consequence of currency inflation or depletion, and from other causes, some other less variable standard of value as a substitute for or to supplement money when functioning as a standard of value, must be found. This is not to be sought among the metals, but rather among the things which in civilized communities are necessary to meet the wants of man. The ideal and what would appear to be the natural unit of this character is that which, in any country, is made up of definite quantities of staples, such as food and fuel, iron, and each of the many other things which a family requires to live in ordinary comfort—each of these several things in fairly close proportion to the family consumption thereof in some unit of time * * *. Let the proposed unit, for example, be called a 'com' (abbreviated from commodity unit), and let it be supposed that to start with the quantities of each of the 250, or more, things which the U. S. Department of Labor takes into account when it fixes the general commodity, 'index numbers' have been determined which, in the proportion of family consumption, would at the average wholesale prices during the ten years 1900 to 1909 (inclusive), have been purchasable for some round number of dollars, say, $1 000. One-thousandth part of these several quantities considered collectively, will then be one 'com'."

It will readily be recognized that if such a unit of value was in common use that it would be incomparably more stable than the dollar. The dollar, as already intimated, has value only because it can be exchanged for things which are necessary or desirable, food, clothing, fuel, education, recreation, and the many other things which

* *Transactions*, Commonwealth Club of California, March, 1920.

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contribute to human comfort and well-being. The "com", on the other hand, represents a fixed measure of the desirable things themselves. Salaries, wages, and rents, and, in short, all obligations to pay at a future time, should be specified in coms and not in dollars. At the consummation of each transaction in coms, however, the settlement would still be made in money. The money equivalent would be ascertainable by a multiplier, the number of dollars then equivalent in value to a com.

Under such a system of evaluating the relative desirability of things and services, each country could proceed independently. The commodity unit could be adapted to the local standard of living. The commodities which taken collectively will best represent value, in the sense of desire to possess, would in each country be selected with a view to their inclusion in quantities bearing a fair relation to their proportional consumption by the family and the individual.

As already stated, money as a medium of exchange, when the commodity unit comes into general use, will be as necessary as heretofore. Would it not be possible, may be asked, to let the com supplant the dollar? This is practically what Professor Irving Fisher is endeavoring to bring about—the stabilization of the dollar. He would have the quantity of gold in the dollar vary to keep pace with the changing value of money. Legislation would be required to accomplish this and complicated government control, the successful attainment of which does not appear to be within easy reach. Nor is it necessary. Alongside of the com as a norm of value, whether legally recognized or not, money would be used as we use it to-day, and would always be specified in transactions which are unaffected by the time element. It would be used in the ultimate consummation of every business transaction.

Much more might be said on this subject which is not new,* but it would carry this discussion too far afield. However, may it not be suggested, that the change in the value of the money unit as measured by the com might, with propriety, be used in some fashion to secure a correlated automatic increase or decrease of public utility earnings. Frequent revaluation at present-day prices might thus be made un-

* The idea of making a commodity unit the standard of value seems to have been suggested as early as 1822, in his "Present State of England," by Joseph Lowe, who pointed out the need for such a standard in long-time contracts. The same idea occurred to G. P. Scrope a few years later and is discussed by him in "Inquiry into the Nature of a Just Standard of Value", and other publications. Professor Jevons discusses these plans in "Money and the Mechanism of Exchange". Among others who have discussed and approved the use of a multiple standard, may be mentioned, Count Soden, Professor Roscher, Professor Walras, F. A. Walker, Simon Newcomb, Alfred Marshall, E. B. Andrews, Thomas E. Will, Henry Winn, Frank Parsons, A. I. Fonda, J. Allen Smith, C. F. Taylor, Eltweed Pomeroy, and Irving Fisher. Some of these economists not only approve the general principle involved, but have advocated the incorporation of the multiple unit into the monetary system, while others would use it in some way to regulate the volume of the circulating medium.

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necessary, particularly if the rate base itself was noted in coms instead of in terms of money.

The writer would suggest in keeping with the foregoing comments that to the glossary of valuation and rate-fixing terms there be added:

Rate Base.—That amount expressed in money on which the owner of income-producing property is entitled to a fair interest return. Preferably, the legitimate investment computed at present-day prices, which the writer would call the "natural rate base."

Going Value.—(As Distinct from Going Concern Value).—That portion of value in excess of the legitimate investment which results from a capitalization of so much of the net earnings as is in excess of fair interest on the natural rate base as previously defined. In this sense, going value will include the value of such intangible elements of value as franchises, water rights, special privileges, and the like, except to the extent that, by reason of investment, these are already made a part of the rate base. It is to be treated as a result of earnings and not as a part of the rate base.

Commodity Unit, or Com, for the United States.—(To Be Hereafter Specifically Defined by Law).—The standard or norm of value whenever in any transaction, such as the fixing of a wage or salary or public utility rates, the time element is material. As a preliminary suggestion, for the United States, the compound commodity equivalent of one dollar during the ten years, 1900 to 1909, inclusive, as determined for these years from the index numbers of economists, based on all commodities, weighted according to their relative annual consumption in the whole country, may be accepted as one com, and the prevailing index number will then establish the value of the dollar in coms for any particular date.

Mr. Mayer.

JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—This paper deals mainly with the difficult problem of the valuation of regulated monopolies.

The valuation and regulation of monopolies are connected so closely that they cannot be investigated independently. Experience has shown that in many important industries, competition—the modern means for securing fair prices—is either impracticable or inadequate to prevent excessive prices and profits. (In this discussion, the word "profit" will stand for "rate of profit"). Regulation of the prices of the products or services furnished by such industries, therefore, was established to fix such prices as will produce the same profits in monopolized and competitive industries.

Two different aims are pursued by the regulating commissions controlling different industries. Some endeavor to secure the same

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average profits in the regulated industry as prevail in otherwise equally attractive competitive industries, while permitting very different profits in individual enterprises. The Interstate Commerce Commission has practically aimed at this policy, or was compelled to follow it by the partly competitive nature of the controlled enterprises.

Other regulating commissions have aimed at standard profits in successful, controlled enterprises about equal to average competitive profits. They fix a permitted maximum profit which may not be exceeded. If, however, the profits attainable by practicable prices are inevitably smaller, the company and not the public has to bear the loss. Successful enterprises which are easily able to earn the maximum allowed profits have then little inducement for avoiding waste by excessive salaries, high costs of materials used, or for the adoption of improvements, since they would have to bear possible losses without securing any profit therefrom in case of success.

The aim of a perfect regulation must evidently be to give to each individual enterprise its savings achieved by high efficiency as well as the losses resulting from inefficiency or bad judgment, while preventing the excessive prices made possible by monopoly. The policy of fixing a maximum permissible profit without regard to efficiency will inevitably frequently lead to waste and retardation of progress. With such a policy, the allowed maximum profit must be larger than the average competitive profits, otherwise the necessary capital for extensions and improvements cannot be secured. To avoid the harmful effects of a fixed maximum profit, some commissions endeavor to fix prices which give with average efficiency about the average competitive profits and provide for sharing between the public and the companies the excess profits secured by superior efficiency or other causes.

The attempt to prevent inefficiency of management leads inevitably to permitting profits varying with efficiency, and a good regulation endeavors to secure average profits in monopolized industries equal to average competitive profits, while allowing profits varying with efficiency in individual enterprises. The existing regulation and the efficiency of management, together with external causes independent of both, have in the past determined the income from and, consequently, the value of the securities of regulated monopolies, and will do so in the future.

The necessity of regulating monopolies so that the capital required for the needed new enterprises and for enlargements and improvements of old ones can be obtained guarantees (with intelligent regulation and in the long run) that average profits in regulated monopolies will be the same as for otherwise equally attractive investments and competitive enterprises. The actual regulations, however, are only rough and fluctuating approximations to those prescribed by this rule.

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The present and the expected future net return to the investor are the main factors that determine the value of the securities. Other factors of importance are: the current rate of return on similar investments, availability for trust funds, steadiness of prices, and ease of selling indicated by frequency of sales on the exchanges. The investor when he wants to sell is interested in the market price; when he wants to buy, he is interested in the present and future market prices and income. If a security is quoted daily on the exchanges, the determination of its present value is simple, since it is given by the quotation unless there is artificial manipulation. The holder is interested in future income and future values, and, therefore, also in the future regulation which greatly influences them; he values a security because it enables him to sell it when he pleases or to obtain an income from it. Everything that changes either the present or future selling price or the income virtually changes the amount of his property.

Radical changes in the regulation of monopolies change both incomes and selling prices of their securities, they either take away or add to the property of their owners and either give or take equal amounts from others without compensation, unless provision is made for the protection of past investors. They are unjust without such provision unless the previous regulation was illegal. The commissions, the legislatures, and the Courts have endeavored to and must be assumed to have given just decisions in the past, unless the contrary is proven by legal proceedings. The values resulting from past legal regulations and those expected in the near future are the actual and fair values of all past investments. The values which would result from a future regulation radically different from that previously expected are neither actual nor fair values. Since any such regulation, unless accompanied by adequate provision to maintain the values of past investments which would have resulted from the previously expected regulation, is unjust, it would probably not be sustained by the Courts.

The author's valuation would evidently give values radically different from those which have resulted from the past and the expected future regulations. It is, besides, so complicated that, except by mere chance, no two independent investigators would obtain approximately the same results; it is, however, an intelligent and interesting endeavor based on a wide and accurate knowledge of economic principles to find what, in the author's opinion, should be the value of regulated companies, but it does not give the actual or fair value of past investments or of existing regulated enterprises. The chance that such a valuation would be approved by the Courts, the legislatures, or the existing commissions is, in the writer's opinion, extremely remote.

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The actual and generally fair values are given by stock exchange quotations whenever reliable quotations are available. For compulsory expropriation, or sales of large blocks of securities, average quotations for a suitable recent period form the best available measure of value unless recent unexpected events largely modifying future income lead to a calculation from estimated future income, based on the observed relation between income and value for similar investments which by past experience have shown approximately the same relation between income and value. Where no reliable quotations are available, a calculation from estimated future income is the best means of ascertaining the value.

Valuations based on cost of production, or of reproductions of the business, or of a similar business, are practically obliged to neglect the effect on its value of efficiency of management, of unexpected outside factors, and of regulation if any exists, because this can only be measured by the income they produce. The value of a property depends mainly on its future income and is measured by normal exchange quotations if such are available. The actual past, present, and the therefrom estimated future income depends not only on costs, but on efficiency in creating and operating the business, on the regulation of prices, on the efficiency of competitors, and on many other factors unforeseen and independent of the management. The combined influence of all of them is measured by the actual income and the consequent quotations. Therefore, there can be no definite relation between costs and values, as correctly observed by the author, but, unfortunately, the same remark applies to the cost of production of a similar prospective competitor, which would indirectly determine the value of any enterprise by his method of valuation.

The regulation must allow monopolies a fair net income on their fair value. To determine their fair total income, their fair value must be known; therefore, it cannot be determined from their income. This is the apparently conclusive argument for the necessity of a valuation which is not based on income, and this explains the numerous efforts to base valuations on costs. The error in this argument must be shown, in order to justify a valuation of monopolies based on income. The present value of a monopoly depends on its present and the estimated future income, and this depends on the present and the expected future regulation. The present regulation is a known present fact, the expected future regulation is a knowable present state of mind. They determine the present and the estimated future income as far as they depend on regulation, and, with other known present facts, the present value of the monopoly. The actual future regulation is unknown and, therefore, has no influence on the present value of the regulated monopoly. The fair present value of the monopoly can be determined

Mr. Mayer. from the known present and expected future regulation, and the fair future regulation can be determined from its present fair value.

The proper guiding principles of a fair regulation of monopolies are, therefore, the following:

1.—The average profits of each monopolized industry, of a fairly uniform economic region, and consisting of many individual enterprises, should be the same as those of similar competitive industries.

2.—The profits of individual enterprises of each monopolized industry should be allowed to vary as those of competitive enterprises.

3.—The future regulation should be based on valuations estimating the total present market value, ascertained from incomes and quotations resulting from the present and the expected future regulation, of all the monopolized enterprises of an approximately uniform economic region, and kept up to date by adding the value of all new investments.

4.—The average rate of profit or ratio of net return to market value in similar competitive industries should be ascertained for each economic region. An accurate determination of actual depreciation is required for determining the real rate of profit in competitive or monopolized industries.

5.—The actual rate of profit in both the chosen competitive and all the monopolized industries should be kept up to date, and such modifications of rates should be prescribed from time to time as to produce equality of profits in monopolies and competitive industries.

It is evident from the foregoing that a valuation based on quotations and actual and estimated incomes can be used successfully as the basis of a just regulation of monopolies and that there is no necessity, therefore, of inventing a far more complicated method which cannot give correct results since it neglects various factors which largely influence actual values.

Mr. Willoughby. J. E. Willoughby,* M. Am. Soc. C. E. (by letter).—The agitation for a valuation of the railways in the United States grew out of the erroneous assumption commonly made that the book value of the railways was greater than the amount of money actually invested in the property. It was also assumed that the railways could be built now for very much less than the actual original cost, due to the advance in the art and appliances of railway construction and due, also, to the greater amount of capital available. It was evident that the values dependent on commission-made rates were not necessarily

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Mr. Willoughby.

the values inherent in the industry, and it was felt that the assumed excess of book values over actual reproduction and development costs were losses to be borne by the industry instead of by the public.

The owners of the railways had definite protection in the Constitution of the United States against the confiscation of their property, and that they should appeal to that protection was natural.

It has never been seriously thought that the cost of reproduction of the physical property of any designated railway company is the fair value of the property of that company. The cost of reproduction of such property is merely one of the elements which go to make up the value of the property; and some of the author's remarks on the cost-of-reproduction method are tinged with error as to the purpose of that method.

The writer understands the author's idea to be: That the fair value of a railway property is the capitalization of the net earnings which the property will produce from a general rate level which is sufficiently high to produce such profit from the construction and operation of a competitor as will attract the capital necessary to create that competitor.

The competitor must necessarily procure some part of the existing traffic, and under the assumptions will not, of itself, produce any additional traffic. If, perchance, the general rate level at the time of the valuation is just sufficient, under normal efficiency in maintenance and operation, to produce an investment interest rate on the book cost of the property under valuation, the net earnings from a "fifty-fifty" division of the traffic, within the "reasonable time which capital would be contented to wait", will require a decided increase in the general rate level, in order to attract the capital necessary for the construction and operation of the competitor. The increase in the general rate level will produce actual profits to the property under valuation, which, when capitalized, will augment the value of that property. The proposed competitor method will give fictitious values to the real property and cannot determine, in practice, the fair value of the property under valuation.

Railways are often built for competitive purposes, but such construction is undertaken when there is expectation of additional traffic to be developed; expectation of additional hauls and revenues from traffic already enjoyed by the building company; expectation of obtaining control of traffic—sometimes called for for strategic purposes; and rarely for the division of existing traffic. The experience of the railways in the United States does not afford the data for valuation by the proposed competitor method. When we depart from experience, we get into a maze of theory which is not an aid in fixing the fair value.

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It is a matter for regret that the fair value of the railways cannot be determined on the principles governing free competition, but the author is entitled to the thanks of the Society for the service rendered in his study relative to the application of those principles.

Mr. Hawley.

R. W. Hawley,* M. Am. Soc. C. E. (by letter).—This paper came rather as a surprise. Even though it proposes the general use of a method of procedure that is in many respects new, there has been such voluminous discussion of the valuation of public utility properties that it seems futile to add anything at this time. It seems doubtful whether the "prospective competitor method of valuation" will actually simplify procedure or result in greater need of justice to all concerned.

It seems to the writer that the author has placed entirely too much stress on the sanctity of private property value. His first basic proposition (page 1314) strikes his keynote. It points, as a thing proven, to the absolute similarity of private property without specific regulation by the State of use, returns or otherwise, and private property called "public utility," recognized by the community as of such general importance that it should be regulated. It is agreed that in this recognition only lies the basic difference in the legal status of properties devoted to the production of anything used by the public.

The writer differs with the statement of this basic principle in that it implies protection of the value of property which is not part of a public utility. If the private owner of a parcel of land must sell, the value of his property is in no way protected by law or the Constitution, while that of a public utility is recognized and has protection far beyond any other property. On the other hand, it is entirely true that the owner of property under no public regulation can sell when and to whom he pleases and the public has no voice in the use to which the property may thereafter be put. It may go from a state of maximum productiveness to a state of absolute idleness. It is due to the fact that the public is vitally interested in the continual operation of a public utility property that there is regulation by public agents under the law and all the details of such regulation are the natural and logical outcome. The difference in status of unregulated private property and public utility property is so absolute and fundamental that the effort of the Courts and Commissions to establish and follow the theory that the property value must be determined as though there were no regulation, has always resulted in approximation without logical bases and argument.

The only basic principle that can be adhered to fully is the assumption from the beginning of control by the public that the owners are agents of the public contracting to provide a certain commodity or service, and that the property involved comprises a site for the con-

* San Francisco, Cal.

duct of the agency and the tools used in producing the commodity or service. Fair payment obviously will then be return upon the actual investment as rental of the property, and the costs of conducting the business and the cost of its upkeep through a depreciation fund. Where there is no record, or records are incomplete, the so-called historical reproduction cost method of determining the amount of investment to be used would naturally follow. Mr. Hawley.

In carrying the regulation of utilities into a time of relatively high prices such as prevail at present, there is no injustice in adding to the former investment under low prices the actual higher recent expenditures in capital. There is obvious injustice to consumers in allowing the agent at this time a return not only upon the actual expenditure at high prices but the elevation to these prices of all property installed at former low prices. This is manifestly profiteering under the protection of the representatives of the public. If such a course is justifiable in the case of public utilities, there should be legislation to increase the face value or the interest rate on all long-term bonds issued prior to 1914, and moreover, this change should be made periodically or continuously to be in conformity with general business conditions.

The author's third basic principle holding that every enterprise under regulation should have the privilege of earning the same return as though it were in the field of free and open competition, lets down the bars to a field of speculative contention larger than any that are open to contestants before regulating bodies even under the cost of reproduction. Assuming the public utility to be a railway system in comparatively flat country where it does not physically become a natural monopoly, it may be possible of theoretical determination within fairly narrow limits. However, such conditions are not common. A single railway built into or through any territory, with the choice of routes and location of stations and terminals, will generally have great advantages over any other possible railway tapping that territory. A water system, particularly in the West, has generally made use of the only logical available water supply and to base the returns to be paid by patrons upon the cost of providing the same service by means of a simulated competing system could very readily result in the payment of several times what would be a fair return upon the investment of the existing utility. This would manifestly be unjust, and in all probability would result in such water rates as would seriously limit the growth of the community or would actually force the abandonment of business at that point. No utility management would agree to such exorbitant rates as could very readily result.

This basic principle may very well be in direct opposition to the author's basic principle No. 4, which states that the utility is not entitled to returns due solely to its monopolistic protection from competition, and the writer submits that any dividing line will be found

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to be very indefinite. The writer is presuming that the author means legal instead of physical protection. The author's method would result in exactly the arguments that have been put forward by many experts for determination of intangible values, including those assigned to water rights, strategically located real property, and going-concern value.

The author's basic proposition No. 6 that the "market value" of any property results from use or usefulness and the measure or amount of profit that is or can be realized, and No. 7, that the fair value (presumably of public utility property) is measured by the actual or possible returns under fair rates, throws the whole question back into the same indeterminate position as a spoke in the "wheel of things" that confounded the first reformers who began regulation. It need not have been repeated that the returns realized, or to be realized, cannot be used as a measure in this adjustment of rates to build up the same income.

To work by deduction from returns to value of property requires analysis of all factors entering into the cost of maintenance and operation, depreciation of properties, etc. The author's method appears to make all of these things a necessary determination, but on purely hypothetical bases. He avoids the only reasonably well defined data—the records of actual cost in connection with the property that is in use. In this whole subject, the questions of equity involved have been steadily clouded by the habit of the Courts in placing great stress upon the term "value" and insisting that an amount be found which would always be called by that name regardless of its use. The disagreement of Courts, lawyers and engineers even in cases where a relatively small amount of money is involved, has shown clearly that there can be no exact determination of the thing called "value" where any part is an estimate. What all Courts and Commissions have at all times been called upon to determine is the amount of investment or capital used for a purpose, and to be recognized in any transaction between the owner and the patrons, the public, or in financial dealings. The sum used need never be called "value" nor confused with the changing prices that are or may be paid for the items that would go to make up a similar plant or one that would provide the same service.

All the author's discussion under general principles and recommended procedure, Sections 4, 5, and 6, is boiled down, at the bottom of page 1323, to the conclusion that fair value and fair rates are interdependent one on the other; that, therefore, one must be determined first and that it is better to first decide upon fair rates. The author's conclusion is presumably influenced by his discussion of a railroad rather than any other public utility, and railroad rates have generally been determined without any reference to cost or value of plant. This has not been true of other public utilities where a compact unit is com-

plete and detached from any other similar unit and where there is no basis for determination of rates without reference to cost, actual or assumed, of the particular plant. Mr. Hawley.

In spite of all theorizing, it is not possible to establish similarity between the regulated public utility property and a business operating under free competition. A public utility is fixed in one place, cannot move its plant and cannot actually go into any other business. There is no general field in its own line to furnish a fundamental level of rates or returns. Particularly in the matter of public water supply, there is certain to be great variation in cost found in comparing communities and no reason why one community very advantageously situated should pay the same water rate as another where physical factors make the cost of water service particularly high; nor should it be possible, if the general water rate is determined by measure of the communities easily served, that the utility in the community where the cost is high should lose money.

It must be agreed that the owner of every regulated enterprise is entitled to earn the same profit as, with the same effort and taking equal risks, he would earn if the money were invested under competitive conditions. The author makes this customary statement, however, with the implication that the tendency is to deny the public utility owner his right and he voices grave warning against this course. As a matter of fact, the owner is usually a stockholder who has made his investment without any particular understanding of the business, and those promoting and operating the utility property are, practically speaking, only employees. The only intelligence exercised by the owner or the stockholder was in determining whether to invest. The energy is that of those promoting and operating, and it is difficult for any one to measure the effort put into the building and operation of any particular property. Return to the owner should be a fair rate of interest on investment, and so far as any argument at this point is applicable, it is the actual investment reasonably made and as extensive as the business warrants. Risk is largely eliminated and the scope of the business can be forecast and provided for more closely than is possible in any competitive undertaking. This makes it entirely reasonable that less than the ruling rate of interest for loan to business with a large element of risk be allowed.

Determination of a plant that could be built to give as widespread and adequate service under the conditions obtaining at the time set may be a fair measure of price to be paid in a transfer. To be fair, this should eliminate from consideration the actual existing plant completely. Once the existing property is recognized, it is necessary to recognize its history. Otherwise, the reasons for construction in the present location and with the existing type of construction would often be an absurdity.

Mr. Byers.

M. L. Byers,* M. Am. Soc. C. E. (by letter).—The writer has read with interest the discussions on his paper. As was to be expected in such an intricate matter, evident misunderstandings have occurred, which a more careful reading of the paper probably will correct.

In the special case cited by Mr. Willoughby, namely, a property on which no increased traffic is expected, the process, in a general way, would be somewhat as follows:

1. Assume the competitor's first year's business, in ton-miles, passenger miles, etc., as one-half that of the property under valuation.

2. Assume this to grow at the normal rate for normal properties of this character, this growth to continue until it equals that of the property under valuation as of such date.

3. In the absence of known unwise construction, take the construction plus additions and betterments as equal to the cost of reproduction, at present-day prices, of the physical property under valuation.

4. Find the rate level which, in the time allowed, would just give a fair rate of return on the cost to date of the competitor.

5. Capitalize the net earnings of the property under valuation on the basis of this rate level. The result is its fair value.

This process would involve little more than finding "cost of reproduction"; also, it would not necessarily require a decided increase in the general rate level as surmised by Mr. Willoughby. On the contrary, if the property under valuation had been built when costs were higher, it might call for a lowering of the rate level.

Realizing that opinions of what is fair will always differ more or less, the writer has abandoned opinions of "fairness" as the foundation for his paper and instead has founded the argument on the proposition stated at the top of page 1325 that, "the owner of every regulated enterprise is entitled to earn the same profit as the same amount of capital, intelligence, and industry would normally enable him to earn in the field of free and open competition." He offered in defense thereof the foot note on the same page.

To the writer, the crux of this valuation question is outlined in the brief summary of his paper. How is one to distinguish between a property abnormally well conceived, built, and operated and one abnormally poorly so, in order that he may reward the one and penalize the other? In the questions, *A* to *H*, inclusive, pages 1352-1354, he had hoped to elicit some discussion of these features.

The Prospective Competitor Method seems readily to furnish the answers to these questions.

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Paper No 1454

A SIMPLE METHOD OF COMPUTING DEFLECTIONS OF A CABLE SPAN CARRYING MULTIPLE LOADS EVENLY SPACED

By F. C. Carstarphen, M. Am. Soc. C. E.

With Discussion by Messrs. William Hewitt, Edward B. Durham, and F. C. Carstarphen.

Synopsis.

This paper presents a simple method for computing the deflections of multi-loaded spans when the loads are spaced uniformly that can be applied to those calculations which are involved in determining the length of suspender rods for suspension bridges, the position of the rolling loads on aerial tramway cable spans, and the construction of messenger or electric transmission lines.

First, a general description of the method of computation and its dependence on the theory of first and second differences is presented; then the derivation of the several formulas involved, the proof of the general case of a span the terminals of which are at different elevations and carrying n loads, and the solution of practical examples are given.

It is believed that the method is so simple as to appeal to all engineers who have occasion to make calculations of this nature.

The late Dr. Alexander Macfarlane delivered at Lehigh University a series of lectures on the prominent British mathematicians and physicists of the Nineteenth Century. Among others, he referred to Charles Babbage in the following anecdote:

Babbage was sitting one evening in the rooms of the Analytical Society at Cambridge, in a dreamy mood, with his head leaning forward on the table and a table of logarithms lying open before him. Another member entering the room and seeing him thus occupied, said: "Well, Babbage, what are you dreaming about?"; to which he replied: "I am thinking that all these mathematical tables might be calculated by machinery." This chance remark evidently caught his fancy, for it shaped in a large measure the direction of his subsequent career. He became intensely interested in methods for reducing the details of computation, and for this purpose gave his attention to the theory of finite differences.

Babbage is probably best known to the world at large by his ingenuity in preparing the drawings for and partly building machines called the Difference and Analytical Engines. The British Government is said to have subsidized the construction of the Difference Engine at a cost of more than £17 000. It is generally conceded that the principles established by Babbage in these devices pointed the way to the development of the modern computing machines.

The fundamental idea underlying the method of differences may be illustrated by the following properties of the squares of the natural numbers:

Numbers	1	2	3	4	5	6	7	8	9
Square	1	4	9	16	25	36	49	64	81
1st Differences	1	3	5	7	9	11	13	15	17
2d Differences	2	2	2	2	2	2	2	2	2

For instance, if we write the squares under the numbers, we can find their first difference by subtraction. The second differences are found also by subtraction of the first differences and, in this case, all are constant.

For the sake of illustration let it be assumed that the computation of a table of squares of the natural numbers is a complicated process and that we are endeavoring to discover a method of simplifying it.

Upon an examination of the table of squares and their first and second differences, it will be noted that it is only necessary to add to a square the preceding first and second differences to secure the next square: thus, $9 + 5 + 2 = 16$, or $36 + 11 + 2 = 49$, noting that this is a summation in a diagonal direction. Surely no one would criticize

a computation of this sort because of its complexity. This method may often be used to advantage in computing logarithms and other transcendental functions where the nth difference is not equal to zero, for where a certain number of decimal places only are required, it is possible to secure a difference which is sufficiently near zero to make practicable the use of the method of differences.

It seems that most engineers believe that the properties of curves can only be ascertained by difficult methods of computation. The calculation of the deflections of multiple-loaded cable spans is no exception. The usual formulas given in engineering handbooks for this purpose are either very complicated or are based on the assumption that the concentrated load can be considered as uniformly distributed, and so the practice has become more or less general of adopting graphical methods for solving the problem. The construction of an equilibrium polygon for a large number of loads requires time and skill. The method which follows is so simple and direct that it should be known to all engineers who have occasion to deal with problems of this nature.

The curve assumed by a cable supporting multiple loads is not to be considered as a continuous function, but, on the contrary, consists of $n + 1$ parabolas which intersect beneath the successive loads with an angle the tangent of which is equal to $\frac{g}{t}$, or the weight of the load, g, divided by the horizontal component of the tension in the cable, t. Because of this lack of continuity, all equations for direct computation of deflections are complex.

Strange as it may seem, when the nature of the curves is considered the second differences are found to be constant, similar to the case for the table of squares, and their value is equal to $-\frac{a}{t}(g + w\,a)$, in which

a = Horizontal distance between loads;
t = Horizontal component of cable tension;
g = Weight of load;
w = Horizontal projected weight per foot of cable.

When an odd number of loads are symmetrically placed on the cable with regard to the center of the span, the first differences, exclusive of the slope increment, are equal to one-half of the second differ-

ences; that is, the first differences are $-\frac{a}{2t}[g + w a] \pm q h$, where $q = \frac{a}{s}$, or the load spacing divided by the span.

As this latter case is the one which most frequently arises in practice, the computation of the deflections of such a span by the method of finite differences becomes nearly as simple as the calculation of the table of squares previously given.

Derivation of the Formulas Used.

The derivation of the formula for the deflection of any point along a suspended cable carrying n equal loads evenly spaced, is as follows: To find the deflection at any point P, Fig. 1:

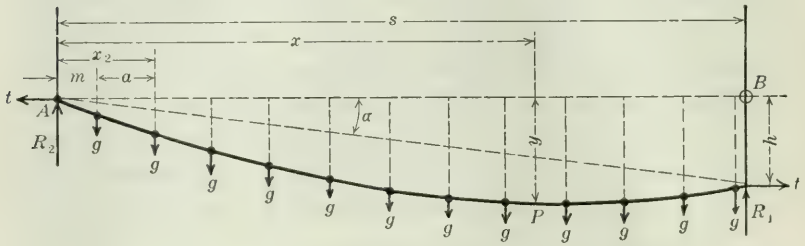

Fig. 1.

Let n = Number of loads on the span;
a = Distance (horizontal) apart or spacing of loads;
m = Distance (horizontal) from R_2 to first load;
h = Difference in elevation of supports;
α = Slope angle;
s = Span (measured horizontally);
t = Horizontal component of the tension;
p = Number of loads to left of x y, or P;
x = Horizontal distance from left support to P;
y = Deflection referred to left support;
R_1 = Reaction at right support;
R_2 = Reaction at left support;
w = Weight per unit of cable multiplied by sec. α;
g = Weight of concentrated load.

For equilibrium:

$$\Sigma x = 0 \quad \Sigma y = 0 \quad \Sigma \text{ moments} = 0$$

that is:

$$\Sigma x = t - t = 0$$

$$\Sigma y = (R_1 + R_2) - (n\,g + w\,s) = 0$$

and by moments about B:

$$R_2\,s = (s - m)\,g + (s - m - a)\,g + (s - m - 2a)\,g + (s - m - 3a)\,g$$

$$\ldots + [s - m - (n - 1)\,a]\,g + \frac{w\,s^2}{2} + h\,t;$$

that is:

$$R_2 = n\,g - \frac{n\,m\,g}{s} - \frac{n\,(n-1)\,a\,g}{2\,s} + \frac{w\,s}{2} + \frac{h\,t}{s}.$$

Consider any free section, such as to the left of P, that is, P is between the pth and $(p + 1)$th loads.

Then,

$$y\,t = x\,R_2 - \frac{w\,x^2}{2} - (x - m)\,g - (x - m - a)\,g \ldots [x - m - (p-1)\,a]\,g$$

and

$$y = \frac{x\,R_2}{t} - \frac{w\,x^2}{2\,t} - \frac{p\,x\,g}{t} + \frac{p\,m\,g}{t} + \frac{p\,(p-1)\,a\,g}{2\,t}$$

$$y = \frac{x}{t}\left(n\,g - \frac{n\,m\,g}{s}\right) - \frac{n\,(n-1)}{2} \cdot \frac{a\,x\,g}{t\,s} + \frac{w\,s\,x}{2\,t} \pm x \tan.\,\alpha - \frac{w\,x^2}{2\,t}$$

$$- \frac{g}{t}\,p\,x + \frac{g}{t}\,p\,m + \frac{p\,(p-1)}{2}\,\frac{g\,a}{t}$$

Hence is derived the general equation, as follows:

$$y = \frac{g}{t}\left[x\,(n - p) - m\left(\frac{x\,n}{s} - p\right) - a\left(\frac{b\,x}{s} - c\right)\right]$$

$$+ \frac{w\,x}{2\,t}\,(s - x) \pm x \tan.\,\alpha,$$

where $b = \dfrac{n\,(n-1)}{2}$, $c = \dfrac{p\,(p-1)}{2}$, and the $\pm$ sign is used to indicate the direction of the slope.

Since the quantities entering into the foregoing equation are usually of large magnitude, it is often preferable to use the following modified form:

$$\text{Let } \frac{x}{s} = r;$$

$$\frac{m}{s} = r;$$

$$\frac{a}{s} = q.$$

then

$$y = \frac{g\,s}{t}\left[(n-p)\,r - v\,(n\,r-p) - q\,(b\,r-c)\right] + \frac{w\,s^2}{2\,t}\,r\,(1-r) \pm r\,h.$$

With a single load only at the center of the span, the deflection at the load is

$$y = \frac{g\,s}{4\,t} + \frac{w\,s^2}{8\,t} \pm \frac{h}{2}$$

Also, the deflection formula may be altered to give a multiplier for $\frac{g\,s}{4\,t}$ for any number of loads, namely,

Case I.—Odd number of loads, with the middle load placed at the center of the span:

$$y = \frac{g\,s}{4\,t}\left[n - \frac{n^2-1}{2}\,q\right] + \frac{w\,s^2}{8\,t} \pm \frac{h}{2}$$

Case II.—Even number of loads, one of the middle loads placed $\frac{a}{4}$ beyond the center of the span. The deflection at this load is:

$$y = \frac{g\,s}{4\,t}\left[n - \frac{n^2}{2}\,q + \frac{n}{4}\,q^2\right] + \frac{w\,s^2}{2\,t}\left[\frac{1}{4} - \frac{1}{16}\,q^2\right] + \left[\frac{1}{2} \pm \frac{1}{4}\,q\right] h$$

Case III.—An even number of loads placed symmetrically to the center of the span. The deflection of the cable at the center of the span is:

$$y = \frac{g\,s}{4\,t}\left[n - \frac{n^2}{2}\,q\right] + \frac{w\,s^2}{8\,t} \pm \frac{h}{2}$$

The following tabulation for an odd number of equal loads, Case I, gives the center deflection in terms of q:

No. of loads.	Deflection due to load.	Deflection due to cable.	Quantity due to chord slope.
1.....	$\frac{g\,s}{4\,t}$	$\frac{w\,s^2}{8\,t}$	$\pm\frac{h}{2}$
3.....	" (3-4q)	"	"
5.....	" (5-12q)	"	"
7.....	" (7-24q)	"	"
9.....	" (9-40q)	"	"
11.....	" (11-60q)	"	"
13.....	" (13-84q)	"	"
15.....	" (15-112q)	"	"
17.....	" (17-144q)	"	"

No. of loads.	Deflection due to load.		Deflection due to cable.		Quantity due to chord slope.	
19.....	$\frac{g\,s}{4\,t}$	(19-180q)	....	$\frac{w\,s^2}{8\,t}$	..... $\pm$	$\frac{h}{2}$
21.....	"	(21-220q)	....	"	.....	"
23.....	"	(23-264q)	....	"	.....	"
25.....	"	(25-312q)	....	"	.....	"
27.....	"	(27-364q)	....	"	.....	"

To illustrate the method of determining the center deflection by these formulas, take, for example, the following span:

$s = 2\,000$ ft.

$h = -200$ ft., that is, right support is above left one.

$g = 2\,000$ lb.

$w = 3.6$ lb-ft.

$a = 204$ ft.

$q = \frac{a}{s} = 0.102$

$t = 40\,000$ lb.

It will be noted that when $a = 204$ ft., it is possible to have the following arrangement of loads:

Case I.—$n = 9$, with fifth load at center of span.

Case II.—$n = 10$, with sixth load 1 051 ft. from left support.

Case III.—$n = 10$, with sixth load 1 102 ft. from left support.

Therefore:

Case I.—

$$y = \frac{2\,000 \times 2\,000}{4 \times 40\,000}[9 - 40 \times 0.102] + \frac{3.6 \times 2\,000^2}{8 \times 40\,000} - \frac{200}{2}$$

$= 25 \times 4.92 + 45 - 100$

$=$ 68 below left support.

Case II.—

$$y = \frac{2\,000 \times 2\,000}{4 \times 40\,000}[10 - 50 \times 0.102 + 2.5 \times 0.102^2] + \frac{3.6 \times 2\,000^2}{8 \times 40\,000}\left[1 - \frac{0.102^2}{4}\right] - 200\left[\frac{1}{2} + \frac{0.102}{4}\right]$$

$= 25 \times 4.926 + 44.9 - 200 \times 0.5255$

$= 123.15 + 44.88 - 105.10$

$=$ 62.93 below left support.

Case III.—

$$y = \frac{2\ 000 \times 2\ 000}{4 \times 40\ 000}[10 - 50 \times 0.102] + \frac{3.6 \times 2\ 000^2}{8 \times 40\ 000} - \frac{200}{2}$$

$$= 25 \times 4.90 + 45 - 100$$

$$= 67.5 \text{ below left support.}$$

PROOF THAT SECOND DIFFERENCES ARE CONSTANT.

To prove that the second differences of consecutive deflections of equal loads and empty cable of a suspended span are constant, let the following formula give the deflection referred to the left-hand support for any point, *xy*, Fig. 2:

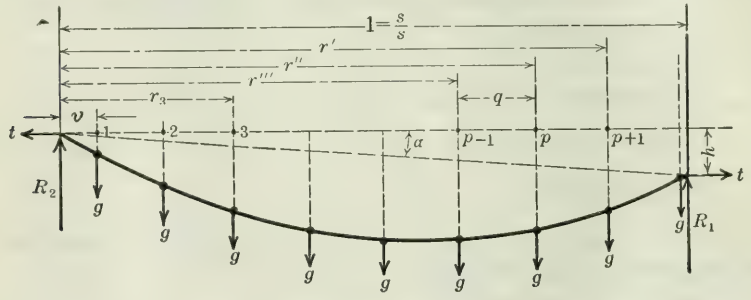

FIG. 2.

$$y = \frac{g\,s}{t}[(n - p)\,r - v\,(n\,r - p) - q\,(b\,r - c)] + \frac{w\,s^2}{2\,t}\,r\,(1 - r) \pm r\,h,$$

where the notation is as before and

r = ratio of distance, x (measured from left support), to span, s, that is, $r = \frac{x}{s}$;

v = ratio of distance, m, to s, that is, $v = \frac{m}{s}$;

q = ratio of load-spacing, a, to s, or $q = \frac{a}{s}$;

$b = \frac{n(n-1)}{2}$;

$c = \frac{p\,(p-1)}{2}$.

Before the value of the second differences of consecutive deflections can be found, it is necessary to obtain their first differences; thus, the first differences between deflections at any consecutive loads, such

as p, $p-1$, and $p+1$, can be determined as follows, if for convenience r', r'', and r''' be the distance ratios to the loads, $p+1$, p, and $p-1$.

If y', y'', and y''' represent the deflections, we may write, noting that $r''=(r'-q)$, $r'''=(r'-2q)$, and $(r'-v)=(p-1)\,q$:

$$y'=\frac{g\,s}{t}\left[(n-p)\;r'-v\,(n\,r'-p)-q\left\{\frac{(n-1)}{2}\,n\,r'-\frac{p\,(p-1)}{2}\right\}\right]$$
$$+\frac{w\,s^2}{2\,t}\,r'\,(1-r')\pm r'\,h\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

$$y''=\frac{g\,s}{t}\,[n\,(r'-q)-p\,(r'-q)-v\,n\,(r'-q)+v\,p-q\,b\,(r'-q)+q\,c]$$
$$+\frac{w\,s^2}{2\,t}\,(r'-q)-(r'-q)^2\pm(r'-p)\,h\ldots\ldots\ldots\ldots(2)$$

Subtract Equation (2) from Equation (1):

$$(y'-y'')=\frac{g\,s\,q}{t}\,[n-p-v\,n-q\,b]+\frac{w\,s^2\,q}{2\,t}\,(q-2\,r'+1)\pm q\,h\ldots(3)$$

The last term is $\pm$, because the slope may be positive or negative.

Equation (3) is very important, as it permits ready computation of the change in deflection between adjoining loads.

To find the second differences:

$$y'''=\frac{g\,s}{t}\left\{n\,(r'-2\,q)-(p-1)\,(r'-2\,q)-v\,n\,(r'-2\,q)\right.$$
$$\left.+v\,(p-1)-q\,b\,(r'-2\,q)+q\left(\frac{(p-1)(p-2)}{2}\right)\right\}$$
$$+\frac{w\,s^2}{2\,t}\,[(r'-2\,q)-(r'-2\,q)^2]\pm(r'-2\,q)\,h\ldots\ldots\ldots(4)$$

Subtract Equation (4) from Equation (2):

$$(y''-y''')=\frac{g\,s\,q}{t}\,[n-p+1-v\,n-q\,b]$$
$$+\frac{w\,s^2\,q}{2\,t}\,[1-2\,r'+3\,q]\pm q\,h\ldots\ldots\ldots\ldots\ldots(5)$$

Subtract Equation (5) from Equation (3):

$$(y'-2\,y''+y''')=-\frac{g\,s\,q}{t}-\frac{w\,s^2\,q^2}{t}=-\left[\frac{g\,a}{t}+\frac{w\,a^2}{t}\right]$$
$$=-\frac{a}{t}\,[g+w\,a]\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

which is the second difference and is a constant.

When an odd number of loads are on the span and the first differences are desired between the deflection of the middle load when

placed at the center of the span and those contiguous to it, use may be made of the fact that under these conditions Equation (3) reduces to

$$y' - y'' = -\frac{a}{2\,t}\,[g + w\,a] \pm q\,h \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

It will be noted that the first part of the second member is one-half the value of Equation (6) for the second difference. Therefore, the calculation of the second difference permits deriving the first differences for Case I in a simple manner. Likewise, for Case III, the first differences are equal to the second differences, slope factors ignored.

EXAMPLES.

The deflection at the loads and the tower heights will be investigated for an assumed span of an aerial tramway across the St. Lawrence River, ends at the same elevation ($h = 0$).

Let s = Horizontal distance between supports = 4 800 ft.
g = Weight of each load = 1 150 lb.
a = Horizontal distance between loads = 375 ft.
t = Horizontal component of tension = 90 000 lb.
k = River clearance = 160 ft.

A.—To find the approximate weight per foot of cable:

Because of the great length of span and the need for keeping the tower heights as moderate as possible, it will be necessary to use a cable of steel capable of working safely under a tension of 67 000 lb. per sq. in. of sectional area. Since the total tension is 90 000 lb., the cross-sectional area of the cable will be $\frac{9}{6.7} = 1.35$ sq. in. Reference to a table of properties of locked-wire cable shows the nearest size to be $1\frac{3}{8}$ in. in diameter, having a listed weight of 4.75 lb. per ft.

B.—It is now necessary to know the number of loads (n) on the span:

The number of loads is one more than the integral portion of the quotient obtained by dividing the span by the carrier spacing; that is,

$$n = \frac{4\,800}{375} + 1 = 13 \text{ loads.}$$

The distance, m, from the left-hand support to the first load, when all are symmetrically placed with regard to the center, is found by

dividing by 2 the difference between the length of the span and the sum of the carrier spacing:

$$m = \frac{4\,800 - (12 \times 375)}{2} = 150 \text{ ft.}$$

C.—To find the deflection under the middle load at the center of the span:

The center deflection for this case of an odd number of loads ($n = 13$) is (Case I of the foregoing):

$$y = \frac{g\,s}{4\,t}\,[13 - 84\,q] + \frac{w\,s^2}{8\,t}$$

$$= \frac{1\,150 \times 4\,800}{4 \times 90\,000}\,[13 - 84 \times 0.0781] + \frac{4.75 \times 4\,800^2}{8 \times 90\,000}$$

$$= 98.71 + 152 = 250.71 \text{ ft.}$$

Because of the large value of the center deflection, more correct results can be secured by obtaining the projected weight per foot of cable. This is found by computing the approximate length, l, of the cable in the span as follows:

$$l = s + \frac{8}{3}\frac{y^2}{s}$$

$$= 4\,800 + \frac{2.667 \times 152^2}{4\,800}$$

$$= 4\,812.83 \text{ ft.}$$

The corrected weight of cable per horizontal foot of span is then:

$$\frac{4\,812.83}{4\,800} \times 4.75 = 4.76 \text{ lb.}$$

Since some may question the propriety of using the parabolic formula for the deflection of the unloaded cable, it will be of interest to compute its center deflection by the catenary as well. The parabolic formula is as follows:

$$\frac{w\,s^2}{8\,t} = \frac{4.76 \times 4\,800^2}{8 \times 90\,000} = 152.32 \text{ ft.}$$

A formula for the catenary is:

$$y = \frac{t}{w}\,\text{cosh.}\,\frac{w\,x}{t}$$

$= 18\,947.37 \times 1.00803 = 19\,099.52 - 18\,947.37 = 152.24$ ft.

$x =$ Distance from the center of the span to the point, xy; that is, for the center deflection, x will have a value of one-half the span, or 2 400 ft.

Comparing this value (152.24 ft.) with that obtained by the parabolic formula shows a difference of 0.08 ft., which is a sufficiently close agreement to establish the validity of the calculations made by that formula.

Correcting the value of the center deflection as found in Paragraph *C*, there is obtained:

$$y = 98.71 + 152.32 = 251.03 \text{ ft.}$$

As the river clearance is to be 160 ft., and as the bottom of the bucket of the carrier will be approximately 10 ft. below the cable, the minimum height of the shore towers will be $170 + 251.03 = 421.03$ ft., that is, we may say not less than 425 ft.

D.—To resume the calculation of the deflections, compute the second difference by Equation (6):

$$-\frac{a}{t}[g + w\,a] = -\frac{375}{90\,000}[1\,150 + 4.76 \times 375]$$
$$= -12.229 \text{ ft.}$$

E.—The first difference at the center is equal to the second difference divided by 2:

$$-\frac{12.229}{2} = -6.1145 \text{ ft.}$$

The deflections may now be tabulated as in Table 1. Prepare a column for each of the thirteen loads. Write the center deflection, 251.03 ft., under Load 7. The first difference in deflection between Loads 7 and 6 is — 6.1145 ft. The deflections for Loads 6 and 8 are found by adding this quantity to 251.03 ft., giving 244.92 ft. for each.

The second difference now comes into play and should be entered in the same column as the respective loads. There is now in Table 1 the deflections under Loads 6, 7, and 8, the first differences between Loads 6 and 7 and 7 and 8, and the second differences entered in the load columns.

To find the deflection under Load 9, follow the method shown for computing the table of squares, namely, the algebraic sum of the deflection of Load 8 plus the first and second differences will give the deflection under Load 9, thus:

$$244.92 - 6.1145 - 12.229 = 226.57 \text{ ft.,}$$

TABLE 1.—Deflections of Level Span.

Load.	1	2	3	4	5	6	7	8	9	10	11	12	13
Deflection	30.91	98.17	153.20	196.00	226.57	244.92	251.03	244.92	226.57	196.00	153.20	98.17	30.91
1st Difference.		— 67.259	— 55.031	— 42.801	— 30.573	— 18.344	— 6.1145	— 6.1145	— 18.341	— 30.573	— 42 801	— 55.031	— 67.259
2d Difference.	.....	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	— 12.229	.....	.....

TABLE 2.—Deflections of Inclined Span.

Load.	1	2	3	4	5	6	7	8	9	10	11
Deflection.....	15.20	32.20	39.04	35.72	22.23	— 1.41	— 35.21	— 79.18	— 133.30	— 197.58	— 272.02
1st Difference.....		+ 16.995	+ 6.8354	— 3.3242	— 13.4838	— 23.6434	— 33.803	— 43.9626	— 54.1222	— 64.2818	— 74.4414
2d Difference.....	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	— 10.1596	.....

or, the sum of the first and second difference of Load 7 will be the first difference between Loads 8 and 9. In this case the value is — 18.344 ft. The first difference between the deflections of Loads 9 and 10 will be the sum of the second difference plus the first difference of Loads 8 and 9, namely, — 30.573 ft.

In this manner all the first differences of deflections at the several loads can be computed by addition.

The accuracy of the work may easily be checked by an independent computation for the deflection of any of the loads, preferably the load nearest the support. The formula for the deflection under the load nearest the support is:

$$y = \frac{g\ s\ r}{t}\ [n\ (1 - v) - q\ b] + \frac{w\ s^2\ r}{2\ t}\ (1 - r)$$

in which

$$r = \frac{x}{s} = \frac{m}{s} = v = 0.03125$$

$$y = \frac{1\ 150 \times 4\ 800 \times 0.03125}{90\ 000}\ [13\ (1 - 0.03125) - 6.09375]$$

$$+ 19.04\ (1 - 0.03125)$$

$$= 12.46 + 18.44 = 30.90$$

This checks the value of 30.91 as obtained by the method of differences and shown in Table 1.

As a further illustration of the method, consider the case of an inclined span of $1\frac{3}{4}$-in. locked-coil cable, listed at 7.3 lb. per ft., as follows:

t = 82 000 lb.
s = 1 720 ft.
g = 4 250 lb.
a = 154.4
n = 11
h = Difference in elevation of supports = — 320 ft., that is, the right support is higher than the left.

The problem is to compute the deflection at the several loads with reference to the left support.

A.—To find the approximate weight of cable per foot of horizontal span:

$$\tan.\ \alpha = \frac{320}{1\,720} = 0.1860$$

$$\alpha = 10^\circ\ 32'$$

$$\sec.\ \alpha = 1.0171$$

Therefore, the horizontal projected weight per foot of cable = 7.3 × 1.0171 = 7.42 lb.

B.—To find second difference:

$$-\frac{a}{t}\,[g + w\,a] = \frac{154.4}{82\,000}\,[4\,250 + 7.42 \times 154.4] = -10.1596$$

C.—To find first difference:

$$-\frac{10.1596}{2} \pm \frac{154.4}{1\,720} \times 320 = -5.0798 - 28.7232 = \begin{cases} -33.803 & \text{right} \\ -23.6434 & \text{left} \end{cases}$$

D.—To find the center deflection, for eleven loads:

$$\frac{g\,s}{4\,t}\,[11 - 60\,q] + \frac{w\,s^2}{8\,t} - \frac{h}{2} =$$

$$\frac{4\,250 \times 1\,720}{4 \times 82\,000}\left[11 - 60 \times \frac{154.4}{1\,720}\right] + \frac{7.42 \times 1\,720^2}{8 \times 82\,000} - \frac{320}{2} =$$

$$125.1259 + 33.4623 - 160 = -1.4118 \text{ ft.}$$

That is, the cable at the center of the span is 1.41 ft. above the left support.

Table 2 can now be prepared as a table of deflections, as follows: Eleven columns are numbered 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, and 11, respectively. In Column 6 on the line of deflections enter — 1.41. Between Columns 5 and 6 write the first difference, — 23.6434. Between Columns 6 and 7 write the first difference, — 33.803. In all the columns enter the second difference, — 10.1596. The remaining first differences are found by adding or subtracting the constant second difference, — 10.1596, to — 33.803, and the successive sums or differences obtained.

The deflection for Load 7 is found by adding the first difference to the deflection of Load 6, giving — 35.21. The deflection for Load 5 is found by subtracting — 23.6434 from — 1.41, resulting in the deflection of 22.23. The deflection for Load 8 is found as previously indicated, by the sum of the deflection of Load 7, the first and second

differences; thus, — 35.21 — 33.803 — 10.1596 = — 79.18. In like manner, the deflections for Loads 9, 10, and 11 are obtained.

The first difference between Loads 4 and 5 is found by subtracting the second difference from the first difference of Loads 5 and 6, thus, — 23.6434 less — 10.1596 = — 13.4838.

The deflection of Load 5, 22.23, less — 13.4838 gives 35.72, the deflection for Load 4. In a similar manner the deflections for Loads 3, 2, and 1 are computed.

To check the accuracy of the work the deflection of Loads 11 or 1 may be ascertained independently; thus, for Load 1, which is nearest the left support, the deflection is given by the following formula:

$$y = \frac{g\,s\,r}{t}\,[n \times (1 - v) - q\,b] + \frac{w\,s^2\,r}{2\,t}\,(1 - r) - r\,h$$

$$= \frac{4\,250 \times 1\,720 \times 0.05116}{82\,000}\,[11\,(1 - 0.05116) - 60 \times 0.089]$$

$$+ \frac{7.42 \times 1\,720^2 \times 0.05116}{2 \times 82\,000}\,(1 - 0.05116) - 0.05116 \times 320$$

$$= 13.937 + 6.497 - 16.371 = 15.21 \text{ ft.},$$

thereby checking the deflection at Load 1 within 0.01 ft.

DISCUSSION

Mr. Hewitt.

WILLIAM HEWITT,* ESQ. (by letter).—The writer has been impressed by the thorough and exhaustive manner in which this subject has been treated by the author.

A further proof that the second differences of load deflections in a multi-loaded cable are constant and equal to $-\frac{a}{t}(g + w\,a)$ plus the slope factor, is obtained by considering p as a coefficient of the load spacing, a, having any integral value from 0 up to $n - 1$, or a particular value of the independent variable, x, and, differentiating, writing the general formula thus,

$$y = \frac{g}{t\,s}\left[s\,x\,(n-p) - m\,(n\,x - p\,s) - a\,(b\,x - c\,s)\right] + \frac{w\,x}{2\,t}(s - x) + \frac{h\,x}{s}$$

Making $x = m + p\,a$, we have

$$d\,x = d\,(m + p\,a) = d\,p = (m + p\,a) - (m + p\,a - a) = a$$

and

$$d\left[x\,(s - x)\right] = (m + p\,a)\,(s - m - p\,a) - (m + p\,a - a)\,(s - m - p\,a - a) = a\left[s - 2\,m - (2\,p - 1)\,a\right].$$

In differentiating with respect to p it should be observed that only such terms can be differentiated as represent values of the independent variable, x, and that $c = \frac{p\,(p-1)}{2}$, not being a factor of x, cannot be differentiated and is to be regarded as a constant. As the sum of the coefficients of x for the load expression,

$$s\,(n - p) - m\,n - a\,b = n\left[s - m - \frac{a}{2}(n - 1)\right] - p\,s,$$

we have, for the first difference,

$$\frac{d\,y}{d\,p} = \frac{g\,a}{t\,s}\left[n\left[s - m - \frac{a}{2}(n-1)\right] - p\,s\right] + \frac{w\,a}{2\,t}\left[s - 2\,m - (2\,p - 1)\,a\right] + \frac{h\,a}{s},$$

and for the second difference,

$$\frac{d^2\,y}{d\,p} = -\frac{a}{t}(g + w\,a),$$

which is constant.

* Trenton, N. J.

Mr. Hewitt.

For loads symmetrically placed,

$$m = \frac{s}{2} - \frac{a}{2}(n-1), \text{ and } \frac{d\,y}{d\,p} = \frac{a\,(g + w\,a)}{2\,t}(n - 2\,p) + \frac{h\,a}{s}.$$

For an odd number of loads,

$$p = \frac{1}{2}(n-1) \text{ and } n - 2\,p = 1, \text{ hence, } \frac{d\,y}{d\,p} = \frac{a}{2\,t}(g + w\,a) + \frac{h\,a}{s}.$$

For an even number of loads,

$$p = \frac{n}{2} - 1 \text{ and } n - 2\,p = 2, \text{ hence, } \frac{d\,y}{d\,p} = \frac{a}{t}(g + w\,a) + \frac{h\,a}{s}.$$

For an odd number of loads, therefore, the first difference is one-half the second plus the slope factor, and for an even number of loads it is equal to the second plus the slope factor.

The writer has had occasion to use this method of determining the load deflections of a cable for a multi-loaded span, and has found it a great saving in time and labor, as compared with the method heretofore used of applying the general formula for each load, and that errors in the computations are less likely to occur. He has also, in a few instances, checked his figures by the application of the general formula, and has obtained the same results, or results with insignificantly small differences.

The discovery by Mr. Carstarphen that the second differences in the deflections of a multi-loaded cable are constant, and can be computed so readily and quickly, is a surprising one, and a unique and remarkable instance of how the application of an apparently complex formula becomes a comparatively simple operation. This discovery the writer regards as an important one and of far-reaching significance, not only in its bearing on suspension-bridge calculations, but also for the long spans of aerial tramways and other multi-loaded cable spans, where the length and position of the cable must be accurately determined.

Mr. Durham.

EDWARD B. DURHAM,* ESQ. (by letter).—The author has presented a scholarly paper on the subject of cable curves and has called attention to an interesting feature of the equation for a loaded cable, namely, that the second differences of the deflections at successive loads are constant. He has also developed a method of utilizing the feature of constant second differences to calculate these deflections.

The writer has treated this subject of cable curves in Section 26 of Peele's "Mining Engineers' Handbook," and has taken up the case of cables with concentrated loads on pages 1557 to 1559 of that "Handbook". He has determined the deflections for any loading by applying the principle of moments to the particular problem and has considered the method more workable than that given by Mr. Carstarphen. The

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Mr. Durham.

exigencies of a handbook required a terse form of discussion, but an elaboration of the subject may be of interest in connection with this paper.

The position of any cable can be determined by the rule that the deflection at any point of the chord for any loading equals the moment of all the vertical forces on one side of that point divided by the horizontal tension of the cable.

Let s = horizontal span;
h = difference in elevation of the supports;
t = horizontal component of the tension;
w = weight per unit length of cable;
g = weight of one concentrated load;
α = angle of inclination of the chord;

l = length of the chord $= \frac{s}{\cos \alpha}$, or $s \sec \alpha$;

d = vertical deflection of the cable below the chord;
n = number of loads on the span;
e = spacing of the loads;
o = point on the chord from which the deflection is measured;
b and c = horizontal distances, respectively, from the two ends of the span to the point where deflection is measured, $b + c = s$;
R = reaction at one support; and
M = sum of the moments of the vertical forces on one side of Point o.

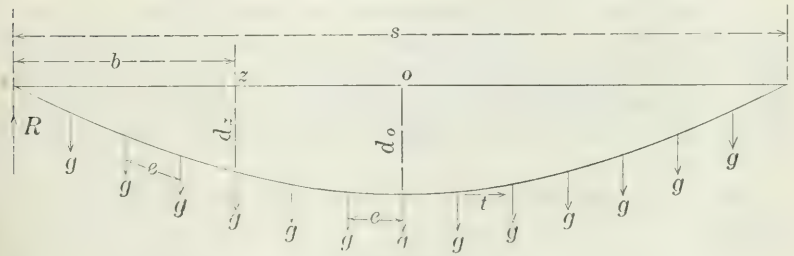

FIG. 3.

Then, in the problem on page 1392, we have (Fig. 3):

$$d = \frac{M}{t},$$

and for the forces to the left of o,

$$M = R\,\frac{s}{2} - g\,(e + 2\,e + 3\,e + 4\,e + 5\,e + 6\,e)\cos\alpha - \frac{w\,l}{2}\,\frac{s}{4}$$

$$R = \frac{w\,l}{2} + \frac{n\,g}{2}$$

Mr. Durham.

then, when, $s = 4\,800$ ft.
$g = 1\,150$ lb.
$t = 90\,000$ lb.
$w = 4.75$ lb.
$e = 375$ ft.
$\alpha = 0$
$h = 0$ ft.
$l = s$.

we have,

$$R = \frac{4.75 \times 4\,800}{2} + \frac{13 \times 1\,150}{2} = 18\,875$$

$$M = 18\,875 \times 2\,400 - 1\,150\,(21 \times 375) - \frac{4.75 \times 4\,800}{2}\,\frac{4\,800}{4}$$

$$= 22\,563\,750$$

$$d = \frac{M}{t} = \frac{22\,563\,750}{90\,000} = 250.71 \text{ ft.}$$

which is close to Mr. Carstarphen's value before correcting for actual length of cable.

Similarly, for another point as z:

$$M = R\,b - g\,(e + 2\,e + 3\,e)\cos\alpha - \frac{w\,b}{\cos\alpha}\,\frac{b}{2}$$

This is applicable to spans with any inclination of chord and with any loading, either uniform or irregular.

For most cable problems, where there are three or more loads uniformly spaced on a span, the calculations can be simplified by using the formula for the deflection of a cable of uniform weight, by making that weight equal to the sum of the actual unit weight of the cable plus a uniformly distributed weight found by dividing the weight of one load by the spacing.

Thus, in the example given previously, the total uniform load per foot is:

$$w_{\text{(total)}} = 4.75 + \frac{1\,150}{375} = 4.75 + 3.067 = 7.817$$

and the deflection at the center of the span is given by

$$d_0 = \frac{w_{\text{(total)}}\,s^2}{8\,t} = \frac{7.817 \times (4\,800)^2}{8 \times 90\,000} = 250.14 \text{ ft.}$$

and at the fourth load from either end by

$$d_z = \frac{b\,e\,w}{2\,t} = \frac{1\,275 \times 3\,525 \times 7.817}{2 \times 90\,000} = 195.$$

Mr. Durham.

Where the span is long and the sag is great, it is desirable to correct the deflections obtained by these methods for the actual length of the cable which may be found by the approximate formula:

$$\text{Length of Cable} = l + \frac{8}{3}\frac{d^2}{l}$$

In the last example, the length

$$= 4\,800 + \frac{8}{3}\frac{250^2}{4\,800} = 4\,834.7 \text{ ft.}$$

The total load on the span and, therefore, the load per foot along the chord, has been increased $\frac{34.7}{4\,800} = 0.724\%$, and, therefore, the deflection will be increased in the same proportion, or:

Deflection at center $= 250.14 \times 1.00724 = 251.96$, and,
Deflection at fourth load $= 195 \times 1.00724 = 196.41$.

These figures are larger than those of Mr. Carstarphen of 251.03 and 196, respectively, as the writer has used the length of the loaded cable and has assumed that the uniform weight on the span due to loading has increased as well as the uniform weight due to the cable.

If the span is inclined, the equation is similar to that given, except that the total load on the span is the length of the inclined chord multiplied by the uniform load per foot and may be written:

$$d_{\text{(at center)}} = \frac{w\,l\,s}{8\,t} = \frac{w\,s^2}{8\,t\cos\alpha}, \text{ and}$$

$$d_{\text{(at any point)}} = \frac{b\,c\,w}{2\,t\cos\alpha}$$

Cable problems like the foregoing usually occur in connection with aerial tramways, and a consideration of their mechanical features will explain the variable and sometimes uncertain character of the initial data and show the necessity of making assumptions to cover some of the conditions.

1.—The tension in the first span is produced by a weight equal to the working tension of the cable. This gives a definite tension in the direction of the tangent to the cable. If the span has an horizontal chord, the tension at the other end of the span will be the same as at the first end, and will be transmitted over the saddle, in the case of a long tramway, to the next span, with a small loss due to the friction on the saddle.

If the first span has an inclined chord and the farther end is higher, the tension at that end will be greater than at the first end; if the farther end is lower, the tension will be less, by an amount equal to the rise or fall multiplied by the unit weight of the cable. This second tension, less the friction on the saddle, becomes the tension

Mr. Durham.

at the beginning of the second span. The tension, t, in the equations is the horizontal component of the tension in the cable at any point and equals the cable tension multiplied by the cosine of the inclination of the tangent to the cable at that point. The exact value of t is, therefore, both unknown and varies from span to span. It is constant for any one span.

In designing a tramway, in order to get a starting point, it is customary to use the working tension of the cable as t and make the layout on that basis, unless it is found that conditions make the tensions in the cable at the end of a span so large as to be unsafe, then the assumed value of t is reduced.

2.—The length of the cable the weight of which produces the sag, is assumed to be equal to the length of the chord, and its weight is assumed to be uniformly distributed along the chord. Neither of these assumptions is exact, but is nearly correct for tramways, as the curves are usually taut and the center deflection is small. The usual formula for the center deflection of a span, *viz.:*

$$d = \frac{w\,l\,s}{8\,t}, \text{ or } \frac{w\,s^2}{8\,t\cos\alpha}$$

where $w\,l$ or, $\frac{w\,s}{\cos\alpha}$, is the total weight on the span, is obtained by taking moments about the center point of the chord, but is based on the previously stated assumptions.

If the span is long or the sag is great, the actual length of the cable may be taken into account.

3.—The loads on a tramway consist of buckets with their contents of freight; these travel on the cable and are spaced and propelled by being gripped to a moving traction rope. They are spaced, therefore, at distances along the cable and are not spaced at uniform horizontal distances, as indicated by the quantity, a, in Mr. Carstarphen's paper, nor at uniform distances along the chord, as assumed by the writer; but the latter assumption is the closer of the two.

As the buckets support the traction rope, on a long span, the weight of each bucket may need to be increased by the weight of a length of traction rope equal to the bucket spacing. On the other hand, the traction rope may be very taut, as occasionally happens, and may tend to lift the buckets and so relieve the cable of part of their weight.

4.—The number of loads on a span can be varied by one and in the operation of a tramway they may occupy an infinite number of positions along the span.

Thus, in the example, which has an horizontal chord, the span, $s = 4\,800$ ft., and the bucket spacing, $e = 375$ ft., assuming that this is also the spacing along the chord, then, $\frac{4\,800}{375} = 12$ spaces $+$ 300 ft.

Mr. Durham.

If one load is at the left-hand saddle, there will be 12 loads on the span and the right-hand load will be 300 ft. from the right-hand saddle. If the loads all move to the right, there will be 13 loads on the span as soon as the load leaves the left-hand saddle, and this number will obtain until they have all moved forward 300 ft. and the right-hand load passes the right-hand saddle. There will then be 12 loads on the span, and this number will obtain while all move forward 75 ft., when another bucket will come on to the left-hand saddle and the initial condition will be restored. During this movement, the cable has been subject to loads in an infinite number of positions and to a variation of one in the number.

Summarizing, the value of t is not known to start with, but is assumed as equal to the working tension of the cable; the weight of the cable and the distribution of the loads are assumed as uniform along the chord, and the number and position of the loads on the span are subject to variation.

In view of these peculiarities of the problems, the writer believes that practical results are obtained by using the total uniformly distributed weight in obtaining the curve where there are at least three loads on the span and these are uniformly spaced. It will be necessary, however, to take the actual loading and solve by using the principle of moments when there are only a few loads on the span or the loads are spaced irregularly. Both these methods of calculating are simple and are easily followed.

Mr. Carstarphen.

F. C. Carstarphen,* M. Am. Soc. C. E. (by letter).—The writer notes with pleasure the comments of Messrs. Hewitt and Durham on his paper dealing with the solution of the multi-loaded cable problem. He appreciates Mr. Hewitt's presentation of a proof of the constancy of the second differences of successive deflections based on the first and second differentials of the modified deflection equation. To many engineers, this proof will appear more elegant than the one presented, based on algebraic subtraction, but it seems that the limits which must be imposed on the value of x of the general deflection formula in order to approach a continuous function of y, and thus permit differentiation to a constant value for the second differential, is more difficult than the proof adopted, unless the reader is more familiar with cable computations than most engineers engaged in general practice.

The writer knows from experience that the method of using the second differences to compute cable deflections is of great practical utility, and hopes that its many advantages will appeal to all who are interested in the solution of problems of this nature. It was thought that the paper presented the subject in a direct and simple manner,

* Trenton, N. J.

Mr. Carstarphen.

but, at this writing, it is not so evident that this aim has been accomplished.

For this reason, the writer is greatly pleased to accept Mr. Hewitt's proof as a strong addition to the argument originally presented, as he is held in high esteem by the Engineering Profession, and has made many contributions to engineering literature. He was associated with the late E. Gybbon Spilsbury, M. Am. Soc. C. E., in developing the manufacture of aerial tramways. It is especially gratifying, therefore, to note Mr. Hewitt's endorsement of the paper as presented, and to learn that he "has found it a great saving in time and labor."

The writer accordingly turns with confidence to a consideration of Mr. Durham's contribution to the discussion. He finds it necessary to declare at once that the paper was not "presented as a scholarly paper on the subject of cable curves," but merely as an explanation of a certain method of computing cable deflections. Mr. Durham appears to make little comment on the paper itself, but offers an apparent defence of some previous exposition of the subject. The writer does not understand the occasion for referring to Peele's "Mining Engineers' Handbook", but it is a pleasure to state that no reflection on it was intended because of the absence of his method from within its covers.

From Mr. Durham's statement that "he has determined the deflections for any loading by applying the principle of moments to the particular problem, and has considered the method more workable than that given by Mr. Carstarphen," it might be inferred that the theory of moments did not enter into the procedure suggested. This, however, is not the case, for it can be readily seen that the formula given on page 1387, namely,

$$y\,t = x\,R_2 - \frac{w\,x^2}{2} - (x - m)\,g - (x - m - a)\,g \ldots . [x - m - (p - 1)\,a]\,g,$$

is merely the general expression for the particular solution given in Mr. Durham's discussion, that is,

$$M = R\,\frac{s}{2} - g\,(e + 2\,e + 3\,e + 4\,e + 5\,e + 6\,e)\cos\alpha - \frac{w\,l\,s}{8},$$

except that the theory of moments is not correctly applied if the value of the load spacing, e, is to have a variable projected length on the horizontal chord, as illustrated in Fig. 3. It may be stated that the equation,

$$y = \frac{g}{t}\,\frac{s}{}\,[(n - p)\,r - r\,(n\,r - p) - q\,(b\,r - c)] + \frac{w\,s^2}{2\,t}\,r\,(1 - r) \pm r\,h,$$

is a summation of all the deflections due to the several individual loads of the span, and will be found to be a most efficient means of computing the deflection at any point of a cable supporting a system of loads uniformly spaced.

Mr. Carstarphen.

It follows from this equation that the second differences of the successive deflections are constant, no matter what position the system of loads may have on the span. It is also a "glimpse of the obvious" that if the loads are spaced the constant distance, a, apart on the horizontal chord, they are also equally spaced when referred to any inclined chord; hence the conclusion reached in the paragraph at the bottom of page 1404 is without weight.

When the system of loads supported by the cable occupies one of the several positions indicated in Case I, II, or III, on page 1388, the general formula is greatly modified, and the computation of the center deflection becomes very short and simple, as will be noted in comparing Mr. Durham's solution on page 1402 of the discussion with that given under C on page 1393.

It does not appear that any further defence of the utility of the method presented need be offered.

If the computation of deflections by the method of reducing the concentrated to a uniformly distributed load offers results which are sufficiently exact for the purpose at hand, well and good, but it is not as accurate as the formula based on the actual loading.

To illustrate: Suppose the cable span referred to on page 1396 carries five loads spaced horizontally 308.8 (a) ft. apart; what is the center deflection? Solving by the formula of Case I, we have the following:

$$y = \frac{g\ s}{4\ t}(5 - 12\ q) + \frac{w\ s^2}{8\ t} \pm \frac{h}{2}$$

$$= \frac{4\ 250 \times 1\ 720}{4 \times 82\ 000}\left[5 - \frac{308.8}{1\ 720}\right] + \frac{7.42 \times 1\ 720^2}{8 \times 82\ 000} - \frac{320}{2}$$

$$= 63.43 + 33.46 - 160 = -63.11 \text{ ft.},$$

that is, above left support. By uniformly distributed load:

$$\left(\frac{g}{a} + w\right)\frac{s^2}{8\ t} \pm \frac{h}{2}$$

$$= \frac{\left(\frac{4\ 250}{308.8} + 7.42\right)1\ 720^2}{8 \times 82\ 000} - = 95.53 - 160 = -64.47 \text{ ft.},$$

above left support, a difference of 1.36 ft.

In so far as the correct deflections may be computed by the method of second differences with the greatest ease for all cases where the number of loads is in excess of three, the writer prefers it to all others.

The remarks concerning the criteria of design for aerial tramways, although of value as an addenda to the section of the Engineers' Handbook previously referred to, do not appear to be germane to the subject-matter of the paper, and accordingly will not be considered here.

Mr. Tarstarphen.

For those who are interested in a formula for computing the deflection of a loaded cable when treated as a catenary, the following is presented:

$$y = \frac{g\,s}{t}\left[(n-p)\,r - v\,(n\,r-p) - q\,(b\,r-c)\right]$$

$$+\frac{t}{w}\left[\sinh\frac{w\,x}{t}\sinh\frac{w\,s}{2\,t} - \cosh\frac{w\,s}{2\,t}\left(\cosh\frac{w\,x}{t}-1\right)\right]\sec\alpha \pm r\,h.$$

When the center deflection of a system of loads is under investigation, such as Case I, on page 1388, this formula reduces to:

$$y = \frac{g\,s}{4\,t}\left[n - \frac{n^2-1}{2}\,q\right] + \frac{t}{w}\left[\cosh\frac{w\,s}{2\,t} - 1\right]\sec\alpha \pm \frac{h}{2}$$

where w is the listed weight per foot of cable, and y is the deflection referred to the left-hand support.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1455

SECOND PROGRESS REPORT OF THE SPECIAL COMMITTEE TO REPORT ON STRESSES IN RAILROAD TRACK*

With Discussion by Messrs. A. W. Buel and E. P. Goodrich.

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November 26th, 1919.

* Presented to the Annual Meeting, January 21st, 1920.

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To the American Society of Civil Engineers:

The Special Committee to Report on Stresses in Railroad Track herewith presents its second progress report.

I.—Introduction.

1.—Preliminary.—As was stated in the first progress report under date of November 3d, 1917, the Committee has been co-operating with the Special Committee on Stresses in Track appointed by the American Railway Engineering Association since the organization of the Joint Committee on June 3d, 1914. As the Committee of the American Railway Engineering Association consists of the members of the Special Committee of the American Society of Civil Engineers, eighteen in all, and three other members of the American Railway Engineering Association who do not hold membership in the American Society of Civil Engineers, the work, with the authorization given by the two Societies, has been carried on as one committee. This report is presented simultaneously to the two Societies. In recognition of the financial support given the Committee, the report is also being presented to the American Railroad Association.

It was early realized by the Committee that an adequate report on stresses in railroad track must be based largely on experimental data derived from extensive tests on standard railroad track, and that in view of the complexity of the action of track under load and the variability of the conditions to be found in track and load the work of conducting experiments and reducing the data would necessarily require a large amount of time and effort. An important element in the work, which has involved expenditure of time, effort, and money, has been the development of methods of conducting the tests and of devising the instruments and apparatus. It has been recognized from the start that in obtaining data on the action of track under the variable conditions of both track and load, great refinement of method was not possible and it was important to make tests under conditions of railroad service as nearly normal as is possible, utilizing also of course the data of laboratory investigations where conditions would not permit satisfactory experiments in the field.

The first progress report of the Committee was published by the Society in Vol. LXXXII., p. 1191, of the *Transactions,* (1918) and also in the *Proceedings* for January, 1918, p. 55; also in Vol. 19 of the *Proceedings* of the American Railway Engineering Association (1918) and in *Bulletin No. 205* of the Association. In this first report the action of the track as an elastic structure is discussed and a method of analysis for determining the effect of single wheel loads and of combinations of wheel loads is developed; the method of conducting the tests and the nature of the instruments used are described; and the

tests to determine the stresses in rail developed by static loads and moving loads under a variety of conditions of track and load are described and the data discussed. In particular, the results with several types of locomotive running at a variety of speeds on rails of several weights are given in some detail. Track depression and rail profiles under a variety of loadings are presented, and the properties of track as an elastic structure determined for a variety of conditions are reported. The report relates principally to stresses in rail and to the general elastic conditions of the track.

Conditions existing during the war greatly impeded the progress of the work of the Committee, and of course only such work was undertaken as could be done without interference with any activity or agency helpful in the prosecution of the war. Even since the war closed the conditions have been very unfavorable for carrying on experimental work. The limitations relate to both the use of railroad equipment and the supply of assistance for continuous work.

The work reported herein includes the results of tests with one type of freight locomotive and one type of passenger locomotive on the St. Louis-San Francisco Railway to determine stresses in rail in relation to speed and counterbalance effect, and also the results of similar tests with one type of freight locomotive on the Illinois Central Railroad. The report gives the results of tests on the Illinois Central Railroad and the Chicago, Milwaukee and St. Paul Railway to determine the depression of track under given loads and to find the depression and flexure of ties and their action under load for a variety of conditions found in track. The tests conducted in the Laboratory of Applied Mechanics of the University of Illinois with a view of finding the methods and principles involved in the transmission of pressure from one or more ties downward and laterally through ballast materials are reported and an analytical consideration of the transmission of pressure is given. The report is presented under the following headings: Tests to Determine the Effect of Speed and Counterbalance on Stresses in Rail; Track Depressions; Depression, Flexure, and Bearing Pressure of Cross-Ties; and Transmission of Pressures in Ballast.

The Committee is continuing work on the subject assigned to it.

2.—Acknowledgment.—Funds for use in carrying on the work of the Committee have been taken from the contributions made by the United States Steel Corporation, the Bethlehem Steel Company, the Lackawanna Steel Company, and the Cambria Steel Company, acknowledgment of which was made in the first progress report. The Committee wishes to express appreciation of the appropriation of a fund by the American Railroad Association for its use in 1919 and 1920.

The co-operation of railroad companies in furnishing facilities for the test work has itself been a large contribution. The Illinois Central Railroad, A. S. Baldwin, M. Am. Soc. C. E., Vice-President, and F. L.

Thompson, Chief Engineer, gave the use of its tracks and of locomotive, cars, and crews for the tests made north of Champaign, Ill. The St. Louis-San Francisco Railway, F. G. Jonah, M. Am. Soc. C. E., Chief Engineer, and V. K. Hendricks, M. Am. Soc. C. E., Acting Chief Engineer, gave the use of track, locomotives, and crews for the tests made near Springfield, Mo. The Chicago, Milwaukee and St. Paul Railway, C. F. Loweth, M. Am. Soc. C. E., Chief Engineer, gave the use of track, locomotives, cars, and crews for the tests at Deerfield, Ill., and Libertyville, Ill.

War conditions have made it difficult to maintain a corps for the work—all who had formerly been engaged on the work went into war service. Mr. H. R. Thomas, who, as Assistant Engineer of Tests, had been connected with the testing operations from the beginning, was in direct charge of the field work in the tests on the St. Louis-San Francisco Railway, but was soon after called into Government war service. Mr. N. E. Ensign, Associate in Theoretical and Applied Mechanics in the University of Illinois, conducted the field tests on the Illinois Central Railroad and the Chicago, Milwaukee and St. Paul Railway and has rendered important service in the supervision of reducing the data on track depression and tie flexure and in interpreting the results, as well as in other parts of the work. Mr. M. L. Enger, Professor of Mechanics and Hydraulics in the University of Illinois, has been responsible for the experimental work on transmission of pressure in ballast; his previous study of transmission of pressure in granular materials brought to this work an experience which has made it possible to carry out the laboratory investigation on transmission of pressure through ballast so satisfactorily. Credit is given Professor Enger for interpreting the data of the tests with ballast and analyzing the problem; he is chiefly responsible for the chapter on "Transmission of Pressure in Ballast". Mr. E. E. Cress, Assistant Engineer of Tests, since January, 1919, has given very efficient service in the preparation of material for the report and in the study and interpretation of data—his thoroughness, familiarity with details, and keen grasp of the problems have been very helpful. Others have assisted in the work from time to time and all have given loyal and careful service.

The University of Illinois has continued to co-operate in the work by giving the use of laboratory, shop and office facilities and through the service of members of the staff of the Engineering Experiment Station from time to time.

II.—Tests to Determine the Effect of Speed and Counterbalance on Stresses in Rail.

3.—Purpose of Tests.—In the first progress report it was stated that analysis alone can give little real information on the important subject of the effect of speed of locomotive and train in increasing the

stresses in track, and that reliance must be placed upon experimental data to determine the influence which speed exerts in this matter. The increase in stress in rail with increase in speed as found from the tests reported was given in a general way for points at and between the wheels of the six locomotives used in the tests. No detailed study of the effect of speed was made, however, and the conditions of the tests were such that the effect of the counterbalancing of the locomotives was masked or not brought out. For simplicity of test and to avoid the introduction of variables which had no bearing on the problems then under consideration, all the runs in the tests reported were made with the counterweight of the front driver in its lowest position as it passed the middle one of the three instruments placed on one rail. The result of this arrangement was that the counterbalance effect existing differed for the several instruments and for the several wheels, but the stresses found at any wheel were approximately the same as the average stress throughout the revolution of the wheel, though it is probable that for some drivers the values found were somewhat larger than this average and for others somewhat smaller. No systematic variation due to position of counterweight was found in the results.

In pursuance of the programme of the Committee, tests have now been made to find the variation in stress in rail throughout the revolution of the drivers and to attempt to separate in some measure the effect of counterbalance from the effect of speed alone. In doing this, it was thought best to include a type of locomotive in which difficulty has been found in placing the desirable amount of counterweight on the main driver but in which, instead, the deficiency of counterweight has been added to the other drivers, making a badly balanced locomotive. The use of this class of locomotives had caused considerable injury to the rail of the railroad company; these locomotives were thought therefore to be extreme in the effects of speed and counterbalance. The counterbalancing of the other types of locomotives used, the Pacific and the Mikado types, is not so difficult a problem in design. The use of the Mikado type locomotive also permitted the tests to be connected up with the tests previously made on the Illinois Central Railroad.

The tests were conducted in the manner described in the first progress report for moving-load tests, the strains in the rail being measured with the stremmatograph.

4.—The Problem of Counterbalancing the Locomotive.—No attempt is made in this report to go far into the analytical side of the problem of the counterbalance and its effect on track. A brief review of the method of calculation commonly used and of the terms herein employed may be of advantage to the reader. To treat the matter in a thorough manner would involve long and complicated mathematical analyses and examples and would require the discussion of a number of elements on which there seems not to be agreement.

The moving driving parts of a locomotive are usually divided into (*a*) reciprocating parts and (*b*) rotating parts. The reciprocating parts include the piston and piston rod, cross-head, and a portion of the main rod (connecting rod). The rotating parts include crank pins, crank-pin hubs, side rods, the portion of the main rod considered to contribute to rotational effects, etc. The motion of these reciprocating and rotating parts produces a dynamic effect on the locomotive and track, and in designing the locomotive, the attempt is made to counterbalance in such a way as to reduce the dynamic effect, as far as may seem practicable, by adding a counterbalancing weight to the drivers at a location opposite the crank pin.

To help in clearness of expression, the following distinction between counterweight and counterbalance will be made in this report. Counterweight will be used as the weight applied or added to the driver at a point opposite the crank pin for the purpose of balancing or helping to balance the rotating and reciprocating parts. The terms underbalance and overbalance will be used for the deficiency and the excess, respectively, of the counterweight as compared with that necessary to balance the rotating parts or to balance the rotating and reciprocating parts. The term counterbalance will be used to indicate the general condition of underbalance or overbalance as it exists in a given driver. The term counterbalancing will be applied to the general problem of endeavoring to balance the rotating and reciprocating parts.

The vertical effect of a lack of balance of the rotating parts is to increase or decrease the pressure of the drivers on the rails and to vary the pressure upward on the equalizer bars and the frame of the locomotive. A lack of balance for the reciprocating parts has little direct effect on the vertical pressures on the rail at the high and low position of the counterweight, though due to the acceleration of the piston and cross-head it may have a modifying influence on the main driver in certain portions of the revolution of the driver. A lack of balance for the combination of reciprocating and rotating parts produces strains in the locomotive frame and horizontal movements of the locomotive such as nosing. Underbalance and overbalance produce similar effects. It is evident that the counterbalance cannot be made to produce both the minimum vertical effect and the minimum horizontal effect. The methods in use attempt to compromise between the two. The practice in locomotive design is to counterbalance for all rotating parts and for a portion of the reciprocating parts, the proportion of the latter varying from one-half to two-thirds. It is evident, then, that so far as vertical pressure on track is concerned, this practice results in general in overbalancing the rotating parts.

In some types of locomotive other difficulties are found. The diameter of the driver may be so small and the weights of the rotating and reciprocating parts so great that there is not space on the main driver

to place the necessary counterweight even if lead is used for the purpose. In such cases it is the practice to place as much counterweight on the main driver as practicable and to make up the deficiency by placing an extra amount of counterweight on the other drivers, dividing this amount equally among these drivers. The result is that the main driver may be underbalanced even for the rotating parts and the other drivers will have their overbalance for rotating parts increased. The effect on the track structure may be very marked, even though the effect on the locomotive itself may not be noticeable.

Another counterbalancing device sometimes used in such a type of locomotive is a counterweight, known as a bob, which is attached to the driver axle inside the bearings. On account of limitations of space in the design of frame commonly used, the radius of rotation of the bob is relatively small, and its weight must be correspondingly large, thus adding to the driver load at low speeds.

The vertical or horizontal pressure developed by the unbalanced rotating and reciprocating parts is called the dynamic augment. This dynamic augment is, of course, a function of the speed, increasing as the square of the speed.

The formula generally used for calculating the dynamic augment is the formula for centrifugal force,

$$C = \frac{W v^2}{g r} \qquad (12)^*$$

in which

W = the weight of the rotating or reciprocating part for which the dynamic augment is desired, or the unbalanced part of the rotating parts in case the resulting rotational effect is wanted;
g = the acceleration of gravity;
v = the rotational velocity or speed of the part;
r = the radius of the circle taken by the center of gravity of the rotating part.

It is convenient to transfer all weights to the equivalent weight at the center of the crank pin and to use the radius of the circle taken by the center of the crank pin relatively to the center of the driver.

The equation implies consistent units, and the values may be taken in pounds, feet, and seconds, with g as 32.2 ft. per sec. per sec. If V is the speed of the locomotive in miles per hour, the speed in feet per second will be $\frac{4.4}{3} V$. The rotational speed of the crank pin v is obtained by multiplying this by the ratio of the radius of the crank-pin circle r

* This equation number follows the last number used in the first Progress Report, *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1206.

to one-half the diameter of the driver. The distance of the counterweight from the center of the driver will be greater than the radius of the crank-pin circle, and a weight of equivalent rotational effect for a radius r may be used, the equivalent weight being found by multiplying the counterweight by the ratio of the rotational radius of the center of the counterweight to that of the crank pin. For determining the vertical dynamic augment, W should include the excess or deficiency of the rotating parts, and should not include the reciprocating parts, a portion of the main rod being taken as a rotating part. The part of the weight of the main rod which should be considered as an equivalent rotating weight at the crank pin may be determined by the principles of mechanics.

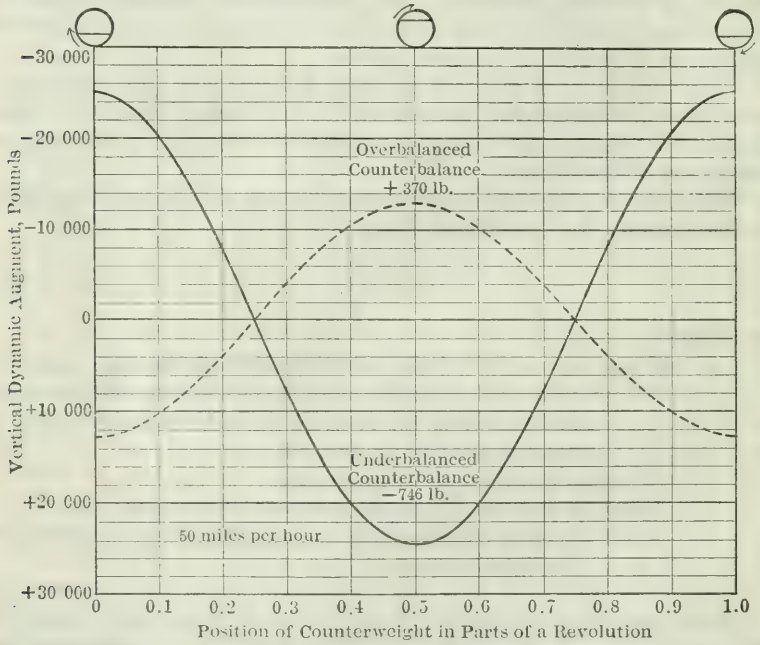


FIG. 1.—VERTICAL COMPONENT OF DYNAMIC AUGMENT FOR OVERBALANCE AND UNDERBALANCE OF ROTATING PARTS.

For determining the horizontal dynamic augment, the weight of the reciprocating parts should be included in determining the unbalanced weight W. In making the calculations, the center of gravity of the rotating part is used in getting the radius of rotation.

Fig. 1 gives, through a complete revolution, the values of the vertical component of the dynamic augment for an underbalance of 746 lb. at the crank-pin circle and also for an overbalance of 370 lb. at the crank-pin circle for a speed of 50 miles per hour, the diameter

of the driver being 60 in. and the radius of the crank-pin circle being one-half the radius of the driver. For the counterweights at one-fourth of a revolution from the low position (90°) the vertical dynamic augment for both underbalance and overbalance is zero, as is that at three-fourths of a revolution (270°). The position giving the maximum downward pressure of an underbalanced counterweight gives the maximum upward pressure of an overbalanced counterweight.

The fact that the center of gravity of the rods and other rotating parts is not in the same vertical plane as the counterweight or the rail is not usually taken into account. The disturbing forces are said to be slight in comparison with the principal rotating and reciprocating forces. The following shows that this element of the problem may need attention.

The plane of rotation of the side rod is some distance outside the plane of the counterweight, the main rod is still farther away, and the part of the pin passing through these rods is also well outside the driver. The result of this eccentricity of position is to increase the effect of the dynamic augment at the central plane of the counterweight, and to require a greater amount of counterweight than would be necessary if these parts and the counterweight were in the same plane. The amount of this increase, in terms of the dynamic augment calculated by the formula given for dynamic augment, is determined by the ratio of the horizontal distance between the center of the counterweight and the center of the rod or other rotating part to the horizontal distance between the centers of the companion drivers. For the locomotives of the three types used in the tests herein reported—the Santa Fe, Pacific, and Mikado—this ratio, found from the dimensions of the parts, averaged about 0.11 for the side rod and the part of the pin within the side rod, and 0.23 for the main rod and the part of the pin within the main rod; that is, the vertical dynamic augment effect at the vertical plane through the center of the counterweight produced by the rotating parts outside of the driver, is 11% greater in the case of the side rod and its part of the pin, and 23% greater in the case of the main rod and its pin, than will exist at the plane of these rods as found by the formula for dynamic augment and centrifugal force. The value of the dynamic augment of these rods and pin calculated by the usual formula must then be increased in the proportion of 11% for the side rod and 23% for the main rod, or the weights of these parts may be considered to be that much larger in determining the underbalance or overbalance.

For the main driver the effect so found is considerable, affecting the amount of counterweight required; a seeming overbalance for rotating parts as judged by the method of calculation generally used in design may become an underbalanced wheel, while for an underbalanced main driver the deficiency in counterweight may be considerably greater than

that found when this eccentricity of position is not taken into account. For drivers other than the main driver the effect is much smaller and may not need consideration.

The foregoing discussion takes the dynamic augment back to the center of the counterweight; the center of the counterweight in locomotives of the types used is generally outside the center of the rail and the point of contact between wheel and rail, and the effect of this is to increase by a small amount the resulting pressure on the rail due to overbalance or underbalance; in the cases under consideration this would probably not increase the dynamic augment by more than 3 per cent. It must be concluded, however, that the location of the rods away from the plane of the counterweight may become of importance in the calculations of the counterbalance of the main driver and should not be overlooked.

The effect due to the rods and the counterweight of a given driver not being in the same plane with one another extends also to the companion driver on the other rail, decreasing the vertical pressure of the companion driver when the pin of the other driver is down and increasing it when the pin is up. It will be seen that the effect upon the companion driver comes at a point in its revolution which is away from the maximum and minimum effect of its own counterbalancing, and hence this element of the problem may not particularly affect the value of the maximum pressure developed by the driver. In case a main driver is balanced for its own masses only, the effect of the rods on its companion driver will be to put it out of balance.

A further condition complicates the problem: the crank pins of the left side of the locomotive are set 90° behind those on the right side. This adds to the stresses in the locomotive and may be expected to produce lateral bending in rails.

In judging of the vertical rail pressure produced by partially balanced rotating parts or by overbalanced rotating parts, it may be sufficiently near the truth to consider that the vertical dynamic augment produced by the excess or deficiency of counterbalance of the rotating parts acts as a vertical pressure on the rail at each driver. This may be expected to be approximately true even when the adjacent drivers have quite different dynamic augment, since the change in depression of the track structure due to the added or decreased pressure on the rail, is small in comparison with the total amount of deflection of the locomotive spring produced by the weight of the locomotive tributary to the given driver, and hence a movement of the wheel downward with the increase in track depression produced by the dynamic augment will result in reducing the load transmitted from the locomotive frame only a relatively small amount and the action of the equalizing levers may be expected to minimize even this effect. It seems proper, then, in making calculations, to consider the vertical dynamic augment of

each driver as being separate downward or upward loads which may be added algebraically to the individual driver loads on the rail. How much this will be modified by the spring of rail and movements of locomotive springs and how much will be transmitted to other wheels by the equalizing levers cannot well be determined by analysis.

Having calculated the vertical dynamic augment of the rotating parts (using the underbalance or the overbalance), the vertical pressures so obtained may be used as positive or negative loads at the wheel points, and calculations for stress in rail may be made for such loads by the methods given in the first progress report of the Committee under "Combination of Wheel Loads".* Illustrations of the use of this method in the calculation of resulting loads and stresses may be found in the discussion of test results which follows.

As the tests for effect of counterbalance were made with the locomotive running with steam shut off, the effect of cylinder pressure does not enter into the results discussed except so far as there may have been resisting pressure developed in the cylinders. For locomotives working under steam pressure, the effect is to increase the downward pressure of the main driver by the vertical component of the main rod tension or compression. For the dimensions of the main rod in the locomotives used this vertical pressure at the up and down points of the crank pin would be from 0.10 to 0.14 of the cylinder pressure at mid-stroke, this vertical pressure presumably being all taken by the main driver.

5.—The Locomotives.—In the tests on the St. Louis-San Francisco Railway two types of locomotives were used, the Santa Fe type of freight locomotive and the Pacific 1060 type of passenger locomotive. Slight variations were made in the locomotives by lightening the reciprocating and rotating parts of the Santa Fe and Pacific types and by changing the position of the equalizer pin of the Pacific type. The locomotives of the Santa Fe type were new, having been in service less than a year. The Pacific type locomotives had been in service about two years.

Fig. 2 gives the spacing of drivers and the wheel loads. The diameter of the drivers in the Santa Fe type was 60 in. and that in the Pacific type 73 in. The crank pin was 15 in. from the center of driver in the Santa Fe type, and 14 in. in the Pacific. The corresponding distance for center of counterweight was 16½ in. in the Santa Fe type and 20¼ in. for the main driver and 25¼ in. for the other two drivers in the Pacific type.

In Table 1 are found the weights of the reciprocating and rotating parts of the Santa Fe locomotive as given by the Mechanical Department of the St. Louis-San Francisco Railway, together with the calculated required counterweights, the amount of counterweight used and

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1208.

the excess or deficiency of counterbalance by the method of calculation used by the Railroad Company. It will be noted that 60% of the weight of the reciprocating parts is included in the counterbalancing calculations and that this weight is divided among four drivers and not among all five, a matter which affects the vertical dynamic augment only as the overbalance or underbalance of the several wheels for rotational parts is affected. Table 1 also gives the excess or deficiency of counterweight needed to balance the rotating parts only, since this excess or deficiency is the principal factor affecting increase or decrease in vertical pressure on the rail.

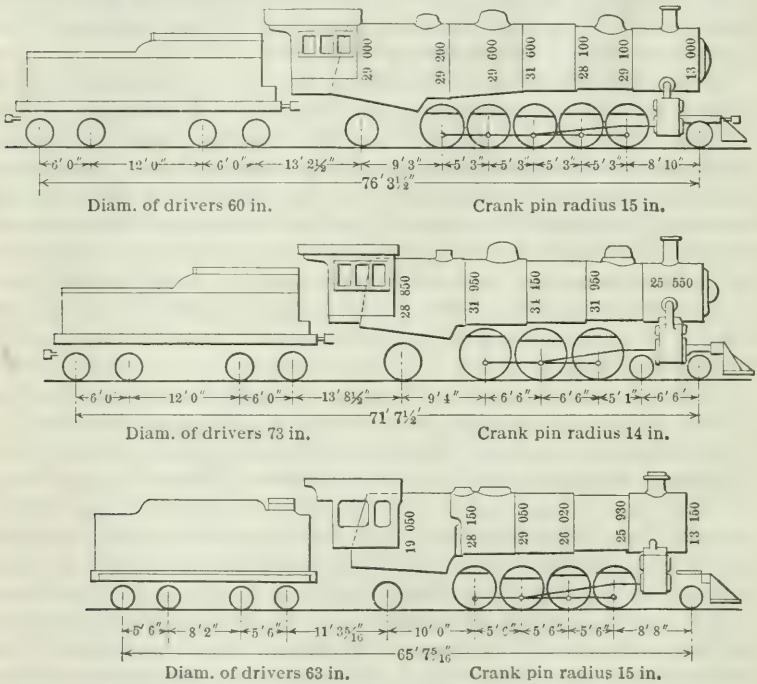


FIG. 2.—DIAGRAMS OF THE THREE TYPES OF LOCOMOTIVES.

There may be of course some question concerning the amount or proportion of the weight of the main rod which contributes to the rotational effect. The excess or deficiency of counterweight needed to balance the rotating parts only, as given in Table 1, is the value usually considered in designing practice, and is found without taking into consideration the effect of the rods and pin not being in the same vertical plane with the counterweight of the driver. The method outlined in the preceding article for taking this variation of position

TABLE 1.—Weights of Rotating and Reciprocating Parts, in Pounds—Santa Fe Type Locomotive of the St. Louis-San Francisco Railway.

Reciprocating Parts.

Piston	803
Cross-head	860
48% main rod, weight on cross-head pin	657
	2 320

$$\frac{0.60 \times 2\,320}{4} = 348$$

Rotating Parts.

Driver number	5	4	Main	2	1
60% of weight of reciprocating parts	348	348	.....	348	348
Main rod, weight on crank pin	.....	.....	735	.....	.....
Side rod, weight on crank pin	190	583	740	504	190
Crank pin	155	162	420	162	140
Crank-pin hub	189	196	305	196	189
One-half eccentric crank	.....	.....	60	.....	.....
Total	882	1 289	2 260	1 210	867
Equivalent weight required at center of counterweight, average taken for two corresponding wheels	794	1 135	2 054	1 135	794
Weight used	812	1 156	1 376	1 156	812
Difference between weight required and weight obtained	+ 18	+ 20	— 678	+ 20	+ 18
Equivalent difference at crank-pin circle	+ 20	+ 22	— 746	+ 22	+ 20
Overbalance or underbalance at crank-pin circle for rotating parts only	+ 368	+ 370	— 746*	+ 370	+ 368

* This becomes 1 042 lb. if the effect of the rods and pin not being in the plane of the counterweight is considered to be as described in the text.

of the dynamic forces into account involved adding to the weight of the side rod and its part of the pin 11% of the weight of these parts as being approximately the effect for the average of the dimensions of the three types of locomotives used in the tests, and to the weight of the main rod used and its part of the pin 23% of the weight of these parts—this increased weight being considered to represent the weight to be used in determining the condition of counterbalance in the plane of the counterweight. Applying these percentages to the parts affecting the main driver, the weight of the main rod coming on the crank pin and that of the side rod and also the parts of the crank pin in the main rod and the side rod, the underbalance of the main driver is found to be 1042 lb. instead of 746 lb.—a very great deficiency. For the other drivers the effect is not so great; there is a reduction in the amount of overbalance of 27 lb. for Drivers 1 and 5 and of 67 lb. for Drivers 2 and 4. It is not known how closely the weights of the locomotives used agree with the weights given in Table 1; the indications are that there are uncertainties in the weights and in their positions.

In Table 2 are given in the same way the weights of the reciprocating and rotating parts of the Pacific type locomotive as furnished by the Mechanical Department of the St. Louis-San Francisco Railway.

TABLE 2.—Weights of Rotating and Reciprocating Parts, in Pounds—Pacific Type Locomotive of the St. Louis-San Francisco Railway.

Reciprocating Parts.

Piston	765
Cross-head	870
Main rod, weight on cross-head pin	535
	2 170

$$\frac{\frac{2}{3} \times 2\,170}{3} = 482$$

Rotating Parts.

	3	Main	1
Driver number			
Two-thirds of weight of reciprocating parts, divided among three drivers	482	482	482
Main rod, weight on crank pin	.....	630	.....
Side rod, weight on crank pin	225	680	224
Crank pin	155	405	150
Crank-pin hub	142	217	142
One-half eccentric crank	.....	60	.....
Total	1 004	2 474	998
Equivalent weight required at center of counterweight, average taken for corresponding wheels	555	1 710	555
Weight used	525	1 605	522
Difference between weight required and weight obtained	— 30	— 105	— 33
Equivalent difference at crank-pin circle	— 54	— 152	— 59
Overbalance or underbalance at crank-pin circle for rotating parts only	+ 428	+ 330*	+ 423

* This becomes 76 lb. if the effect of the rods and pin not being in the plane of the counterweight is considered to be as described in the text.

Applying the method already outlined for obtaining the effect of the main rod and side rod at the plane of the counterweight by means of the factors found from the dimensions of the driver and rods, as just used for the main driver of the Santa Fe type locomotive, it will be found that instead of an overbalance of 330 lb. for the main driver, as given in Table 2, there will be an overbalance of 76 lb., or in effect nearly a balanced condition. The results of the tests indicate an underbalance in the main driver. There appears to be an uncertainty in regard to the accuracy of some of the weights given in this table. It should be noted that the effect upon the counterbalance calculations for the first and third drivers due to the plane of the side rod not being in the plane of the driver is not large, and the error due to neglecting this element is inconsiderable. It may be noted that the larger diameter of the driver of the Pacific locomotive makes the problem of counterbalance easier than for the Santa Fe type even at speeds considerably higher than the freight locomotive would be expected to take.

Fig. 2 also gives the spacing of drivers and the wheel loads of the Mikado locomotive of the Illinois Central Railroad. The diameter of the drivers was nominally 63 in., but the measured circumference was 195 in. Table 3 gives the weights of the rotating parts of the Mikado

locomotives as furnished by the Illinois Central Railroad. Here the consideration of the planes in which the parts rotate reduces the overbalance for rotating parts given for the main driver to what would appear to be an approximate balance. The results of the tests will be found to indicate an underbalance for the main driver instead of a balance or an overbalance.

TABLE 3.—Weights of Rotating Parts, in Pounds—Mikado Type Locomotive of the Illinois Central Railroad.

Driver number	4	Main	2	1
60% of weight of reciprocating parts	263	263	263	263
Main rod, weight on crank pin	.....	630	.....	.....
Side rod, weight on crank pin	145	829	480	154
Total weight considered	408	1 722	743	417
Equivalent counterbalance weight at crank pin in excess of that required to balance crank pin and crank-pin hub.	408	1 722	743	417
Overbalance at crank-pin circle for rotating parts only	+ 263	+ 263*	+ 263	+ 263

* This becomes an underbalance of 15 lb. if the effect of the rods and pin being in the plane of the counterweight is considered to be as described in the text.

It may be of interest to note that the dynamic argument for each pound of underbalance or overbalance considered to be concentrated at the crank pin for the three locomotives used amounts to 33.4 lb. for the Santa Fe type locomotive at a speed of 50 miles per hour, 30.3 lb. for the Pacific type at 60 miles per hour, and 24.5 lb. for the Mikado type at 45 miles per hour; for example, an underbalance of 746 lb. on the Santa Fe type gives a dynamic augment of 24 900 lb.

In all the tests the locomotives were considered to be in good condition. Tires were not worn, except for the main driver of the Santa Fe type (without flanges) which showed more or less wear. The locomotives were representative of those of the same class used on the two railroads.

6.—The Track.—The tests on the St. Louis-San Francisco Railway, made November 20th to December 7th, 1917, were conducted on a section of single track about three miles west of Springfield, Mo. The rail was 85-lb., Am. Soc. C. E. section, rolled in October, 1906. The rail section as worn (shown in Fig. 3) had a moment of inertia (I) of 29.1 in.^{4} about a horizontal axis, and of 6.8 in.^{4} about a vertical axis. The section modulus $\left(\frac{I}{c}\right)$ was 12.0 in.^{3} about a horizontal axis and 2.6 in.^{3} about a vertical axis. The ties were 6 by 8 in. by 8 ft., of oak, spaced about 20 in. apart, center to center, twenty to a 33-ft. rail length. There were 15 in. of chats ballast of good quality. The track was in fair surface and alignment, and was used as it was found. It had not been tamped recently, but as freight trains are not run over this stretch of track the effect of traffic is not great.

The tests on the Illinois Central Railroad were conducted two miles north of Champaign, Ill., September 19th to 21st, 1918, on the southbound track used in the earlier tests reported in the first progress report of the Committee (see page 1237* of the first progress report), the location being that described as Section B. The rail was 85-lb., Am. Soc. C. E. section, laid in 1902. The rail section is given in Fig. 3. The properties used in the calculations were the same as those given in the first progress report, 27.0 in.^{4} for the moment of inertia about the horizontal axis and 11.1 in.^{3} for the section modulus for the remote fiber at the base of rail. The ties were the 6 by 8-in. by 8-ft., oak ties which had been specially prepared for this stretch of track. They were spaced approximately 20 in. apart, 20 ties to a 33-ft. rail. On the section used in the test there was 12 in. of stone ballast. The track had not been tamped for some time and was not in the best condition of line and surface, but it was used in the condition found.

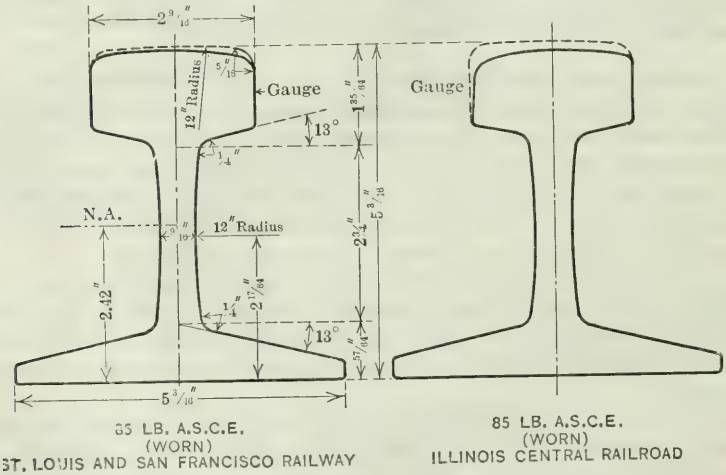


FIG. 3.—SECTIONS OF RAILS USED IN TESTS.

7.—*The Conduct of the Tests.*—The instruments used to determine the strains in the rail were the stremmatographs described in the first progress report, page 1224.* Four stremmatographs were used simultaneously. The methods used were generally the same as those described in the previous progress report. Fig. 4 shows the position of the instruments as used in the tests on the St. Louis-San Francisco Railway. Set-ups *a*, *c*, and *d* were on the same rail, and Set-ups *b* and *e* were on the adjacent rail. It will be noted that in two set-ups, three instruments were on the rail on one side of the track and the fourth

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918).

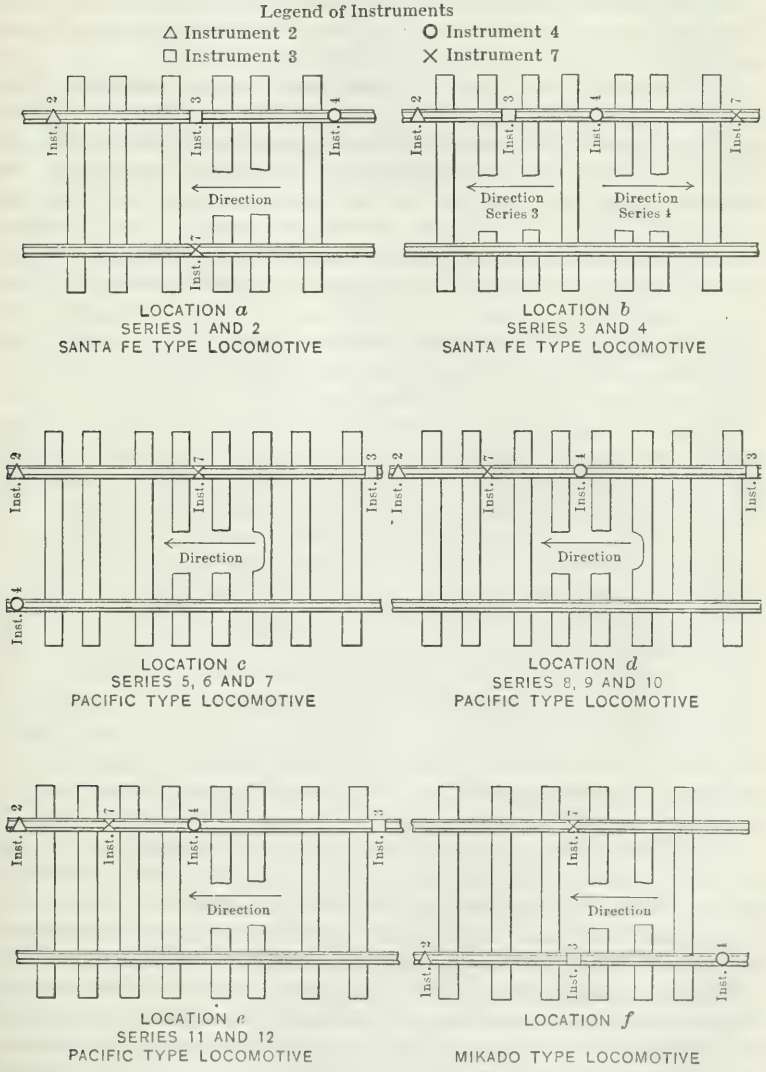


FIG. 4.—POSITION OF STREMMATOGRAPHS IN TESTS.

was on the rail of the opposite side. The instruments were spaced two, three, and four tie spaces apart.

In the tests on the St. Louis-San Francisco Railway, as it was a single-track stretch of railroad, the two sides of the track were presumably the same with respect to ballast and tamping.

In the principal tests made with the Santa Fe type locomotive, three stremmatographs were placed on the rail at the right side of the locomotive and one on the rail at the left side (see Fig. 4, Location *a*). In the tests to determine the stresses developed at the two sides of the locomotive, the four instruments were placed on one rail, and part of the tests were made with the locomotive running in one direction and part in the other (see Fig. 4, Location *b*). In the tests with the Pacific locomotive, the four instruments were placed on one rail (see Fig. 4, Locations *c, d,* and *e*). In all runs steam was shut off as the locomotive approached the test section of track. The speeds were read from a speedometer in the cab connected with the tread of the trailer, the instrument being checked up by timing runs over a measured length of track. For each set of tests the order of speeds for consecutive runs was the same, three or four speeds being used together, and the record of each three or four runs at these speeds was made on one set of disks; new disks were then placed in the instruments.

The position of the counterweight with respect to one instrument was observed for each run. The effort was made to keep the counterweight at the same position with respect to the instrument for the three or four runs covering the three or four speeds used, and then to slip the drivers sufficiently to change the position of the counterweight about one-fourth of a revolution. The slipping of the drivers in stopping and starting, however, changed the position somewhat. The instruments were placed about one driver diameter apart so that for each instrument there was a different position of counterweight. Altogether the tests give a general distribution of position of counterweight over the entire revolution.

The extent of the tests on the St. Louis-San Francisco Railway was limited by reason of the difficulty in sparing the locomotives from service, the demands of traffic at the time being very great. A larger number of runs would have added much to the value of the information. The weather conditions were unfavorable much of the time.

The tests on the Illinois Central Railroad with the Mikado locomotive were made on the south-bound track, and in order to get the necessary distance to acquire high speeds, the locomotive was run northward. Three stremmatographs were placed on the west rail (left side of the locomotive) and one on the east rail opposite the middle one of the instruments. The arrangement is shown at *f* in Fig. 4. The three instruments on the one rail were placed about 63 in. apart, nearly one-third the circumference of a driver.

It should be noted that the rail on which these three instruments were placed was on the outer side of the double track and that, as is shown in the report of the tests on track depression, this side of the track was not in as good surface condition as the inner side. The order of the speeds was 5, 35, 25, and 45 miles per hour, and the four consecutive runs were recorded on the same disks. In all runs steam was shut off as the locomotive approached the test section of track. As with the other tests, the demand of traffic for locomotives was such as to limit the work of the tests. The data secured seem sufficient for general purposes; more tests would have given a wider range of information.

8.—Reduction of Data.—The process followed in the reduction of the data obtained by the stremmatographs in the moving load tests was the same as that which was described in the first progress report. The stremmatograph records were read with a microscope fitted with a micrometer eyepiece. Readings were taken for points in the record corresponding to the position of wheel over instrument and to a point between wheels. In reducing these measurements the readings were multiplied by the proper microscopic constant and then reduced to stresses by multiplying by a constant which involves the value of the moment of inertia and the position of the vertical axis of the rail section, the vertical distance of the needle-bar below the base of rail, the modulus of elasticity of steel (taken as 30 000 000 lb. per sq. in.), and the gauge length (which was 4 in.). A correction was also made to allow for the variation of moment and stress over the gauge length. To obtain the maximum stress at the middle of the gauge length the average stress over the gauge length for points corresponding to the position of wheel over instrument was multiplied by the factor 1.04, as was done with the tests recorded in the first progress report, so that the stresses reported at points of positive moment are stresses in pounds per square inch at the base of the rail at the middle of the gauge length. The variation over the gauge length at points of negative moment was slight and no correction was used for the stresses at such points.

In general, the readings of the two disks of an instrument, giving stresses on two sides of the rail, were averaged and the average was taken as the observation for that instrument for the given run for the given position of wheel. In case the record on one disk of an instrument was defective and that on the companion disk was clear, the one good record was generally also discarded. In case the record of a disk was clear enough to be usable, the record for all the wheels and all the runs on the disk was usually measureable, although the line made by the truck and trailer was less definite than that made by the drivers. Sometimes, however, not all of the record on a disk was clear; in this case the portion available was used. For some of the tests nearly one-

third of the records had to be discarded. It appears that the disk readings taken with the Santa Fe type locomotive were generally clearer than those taken with the Pacific type. The conditions of the tests on the Illinois Central were more favorable and the records were generally good. The records at points of negative moment were generally less clear than those at the wheels, and a larger proportion was discarded.

For any reading or observation, the position of the counterweight of the driver corresponding to the point in the record was found in terms of the part of a revolution of the driver beyond the last down position of the counterweight. This was done for each driver and also for trailer and front truck. In the reduction, the high point of the record was assumed to have been made when the wheel was directly over the instrument.

The readings of the stremmatograph records, the reduction of the data, and the location of the counterweight with respect to each instrument and each point in the record involved a large amount of work, as nearly 100 000 readings were made.

The accuracy of the records and their reduction is considered to be as good as that of the stremmatograph data reported in the first progress report.

It may be noted that for the rail used in the tests on both the St. Louis-San Francisco Railway and Illinois Central Railroad the stress at the extreme fiber of the head of rail is about 10% greater than that in the extreme fiber at the base of rail. The latter has been used in giving results because the width of the base of rail makes the stresses due to lateral bending greater there than in the head of the rail and the lateral bending stresses are great enough to be significant.

9.—Tests on St. Louis-San Francisco Railway.—Table 4 gives information on Series 1 to 12 made on the St. Louis-San Francisco Railway. The injury done to rails by the Santa Fe locomotives soon after they were received from the builders had given the Railroad Company some concern, and the opportunity was taken to make tests with what was evidently a poorly balanced locomotive. The weight on the trailer of the Pacific locomotive seemed excessive, and for a wheel well away from other wheels the stress developed in the rail would be expected to be high. The bearing of the trailer at times had run hot. The tests were made to obtain information on the effect of speed and counterbalance with this type of passenger locomotive.

In Series 1, the order of runs in the tests was 5, 25, and 40 miles per hour for Locomotive No. 22, and 25, 50, 25, and 50 miles per hour for Locomotive No. 28. For Series 2, two sets were run—the first at 5, 25, and 40 and the second at 25, 50, 25, and 50 miles per hour. In Series 3 and 4 the order 5, 25, and 40 was used. For all series with the Pacific type locomotive, the order was 5, 45, and 60 miles per hour.

TABLE 4.—Tests on the St. Louis-San Francisco Railway.

Series number.	Location of test.	Type of locomotive.	Locomotive number.	Position of pin in equalizer bar.	Remarks.
1	First location....	Santa Fe	22. 28	.....	General series with locomotive as received from builders.
2	" "	" "	19	.....	Reciprocating and rotating parts lightened.
3	Second location..	" "	22	.....	Instruments all under right side of locomotive.
4	" "	" "	22	.....	Instruments all under left side of locomotive.
5	First location.	Pacific...	1061	Rear hole.....	Locomotive with stoker added.
6	" "	" ...	1061	Middle hole.....	Do.
7	" "	" ...	1063	Rear hole.....	Rotating and reciprocating parts lightened.
8	" "	" ...	1063	Middle hole.....	Do.
9	" "	" ...	1067	Middle hole.....	General series.
10	" "	" ...	1065	Rear hole.....	" "
11	Second location.	" ...	1066	Middle hole.....	" "
12	" "	" ...	1067	Rear hole.....	" "

The tests were not made in the order of the numbers of the series; instead, the locomotives were taken as the demands of traffic or matters of convenience permitted. In the discussion which follows the series are grouped for the consideration of effect of speed and counterbalance, effect of lightening rotating parts, stress under the two sides of the locomotive, and general comparisons.

10.—The Santa Fe Type Locomotive; Effect of Speed and Counterbalance.—The typical effect of the combination of speed and counterbalance upon stresses in rail for Series 1 with the Santa Fe type locomotives is shown in Fig. 5, which gives sample curves for the main driver taken from the plotted results of the tests with the Santa Fe type locomotive from the data of the instruments on the rail on the right side of the locomotive only. The ordinates of the diagram represent stresses at the base of the rail in pounds per square inch. The abscissas represent the position of the counterweight at the time the wheel passed over the instrument with respect to its down position, the scale being in fractions of a complete revolution of the wheel. The curves represent the average of the results of the several runs at each speed. For a speed of 5 miles per hour the values are represented by a straight line, since the variation from it is small. As the driver is underbalanced, in general the greatest stress in the rail is developed when the counterweight is at its high position. At a speed of 50 miles per hour, it is noted that the maximum value of the curve is 44 700 lb. per sq. in. and the minimum value 14 000 lb. per sq. in. The line marked "mean value of 50 miles per hour curve" is the average ordinate of the curve. The variation above and below this line may be considered to be due largely to the lack of balance of the rotating parts.

The difference between the mean value of the stress found from the curve for 50 miles per hour and the stress at 5 miles per hour may be considered to be largely the effect of speed without reference to effect of rotating parts (quite frequently called impact effect or speed effect), though part may be due to lack of balance of rotating parts. The dotted line of Fig. 5 was obtained by the use of the common formula for the vertical pressure of a rotating part, the calculation here being modified to touch the curve for 50 miles per hour at its high and low points. It will be noted that the form of the observed curves is more pointed than is the curve calculated by analytical methods, and also that

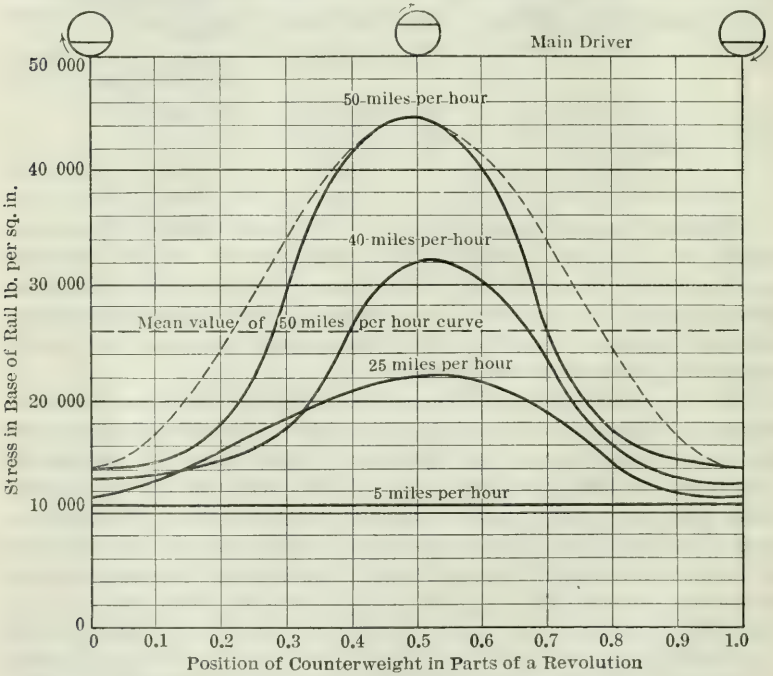


FIG. 5.—STRESS IN RAIL AT MAIN DRIVER OF SANTA FÉ TYPE LOCOMOTIVE THROUGHOUT A REVOLUTION.

the mean value of the curve is generally below the median point of the curve. These differences are characteristic of many of the tests, though it appears that there are as many cases where the observed curve is less pointed than the analytical curve, as for more pointed curves.

As examples of the test data, the test results for Series 1 with the Santa Fe type locomotive for four speeds for the main driver and Driver No. 4, and for one speed for the other drivers and the trailer are plotted in Figs. 6 to 11. Results given by the several instruments are indicated by individual symbols, which are explained in Fig. 4. The curves representing the stress for a given speed throughout an entire revolution

were formed by first averaging the values of the plotted points in groups for each twentieth of a revolution and then drawing a curve to represent as well as possible the position and trend of these points. As in this series there were more runs at 25 miles per hour and less at 50 miles per hour than at the other speeds, the data points for a speed of 25 miles per hour are most numerous, and those for 50 miles per hour the least numerous, the number at the latter speed being only about half that at 5 miles per hour or 40 miles per hour. As may be expected, at a speed of 5 miles per hour the variation in stress due to other causes is so much greater than that due to effect of counterbalance as to mask any effect of counterbalance, and accordingly a straight-line average of all the points has been used for all the wheels for the speed of 5 miles per hour. In each case a line has been drawn above the average curve and one below it which together form the limits of a belt which includes most of the observed points. The distribution of these points vertically over the belt generally is fairly uniform when there are fifty or more points in the diagram. In some cases, with insufficiency of data, the points are quite scattered. The line representing the average ordinate for the curves is marked "mean value of curve"; the line representing the average stress at 5 miles per hour has also been drawn. The diagrams include the results for the instrument on the left rail, although as shown in Article 12 on "Comparison of Stresses under the Two Sides of the Santa Fe Type Locomotive", the stresses in rail on the two sides of the locomotive differed considerably.

Fig. 12 gives the stress in base of rail throughout a revolution of driver for all the drivers and the trailer for Series 1, obtained as described in the preceding paragraph. These curves then represent the average stress for any given position of counterweight. It should be borne in mind that the observed points are not always well distributed in the plots from which these average curves were formed (such as are shown in Fig. 9) and in some cases there is indefiniteness in the data, which makes the forming of the curve difficult and somewhat uncertain. It may be added that the numbers of observed points in the plots are generally not sufficient to establish very definitely the position of the maximum and minimum points on the curve with reference to the position of the counterweight in its revolution.

Fig. 13 gives the stresses in rail under the wheels for the high point and the low point of the curves of Fig. 12. The stresses for negative moment between wheels are also given; for some of the wheels no especially marked difference throughout the revolution was found and the mean of the observed stresses for negative bending moment is reported. Not enough difference was found under the front truck wheel throughout the revolution to warrant giving more than a straight-line average for the stress at this wheel. A similar condition was found under the trailer for a speed of 25 miles per hour, but at the

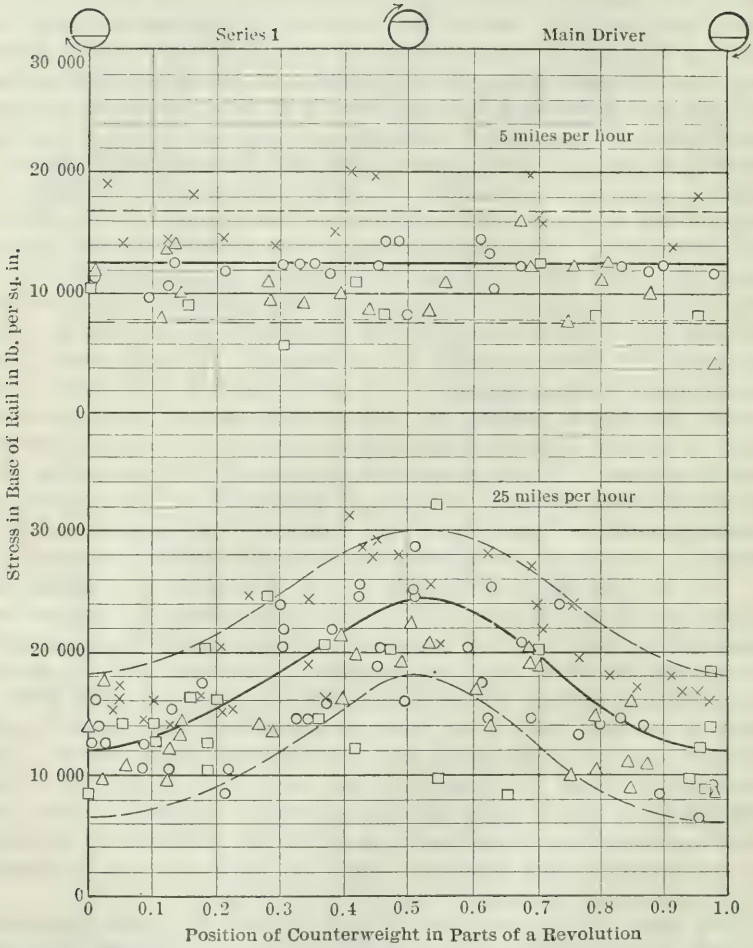


FIG. 6.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

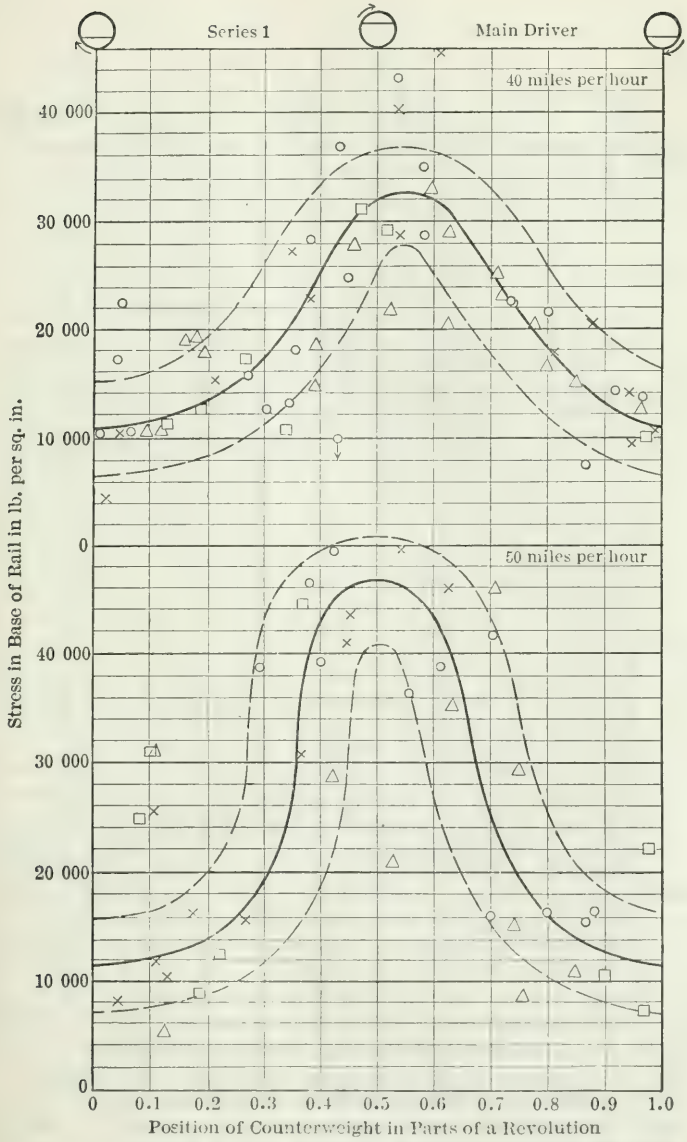


FIG. 7.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

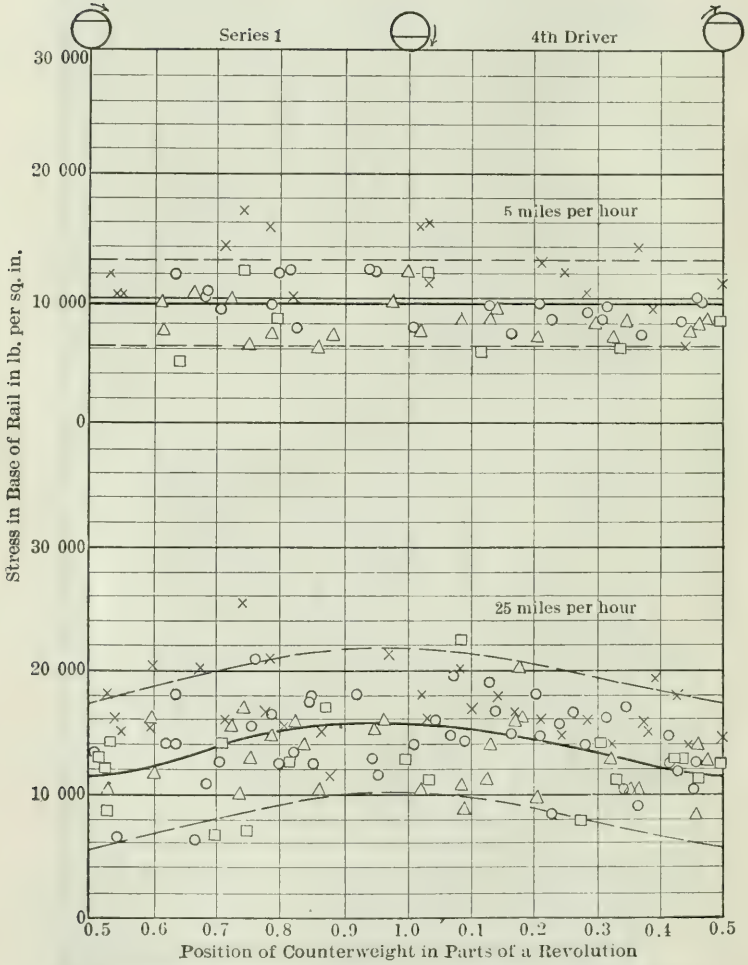


FIG. 8.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

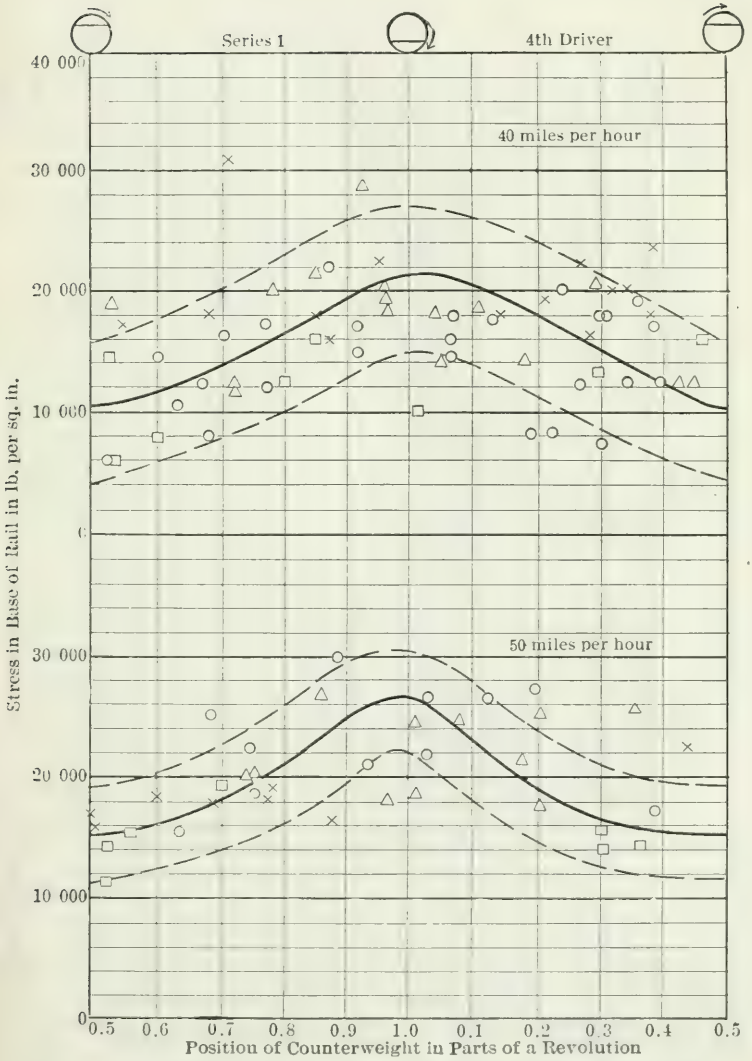


FIG. 9.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

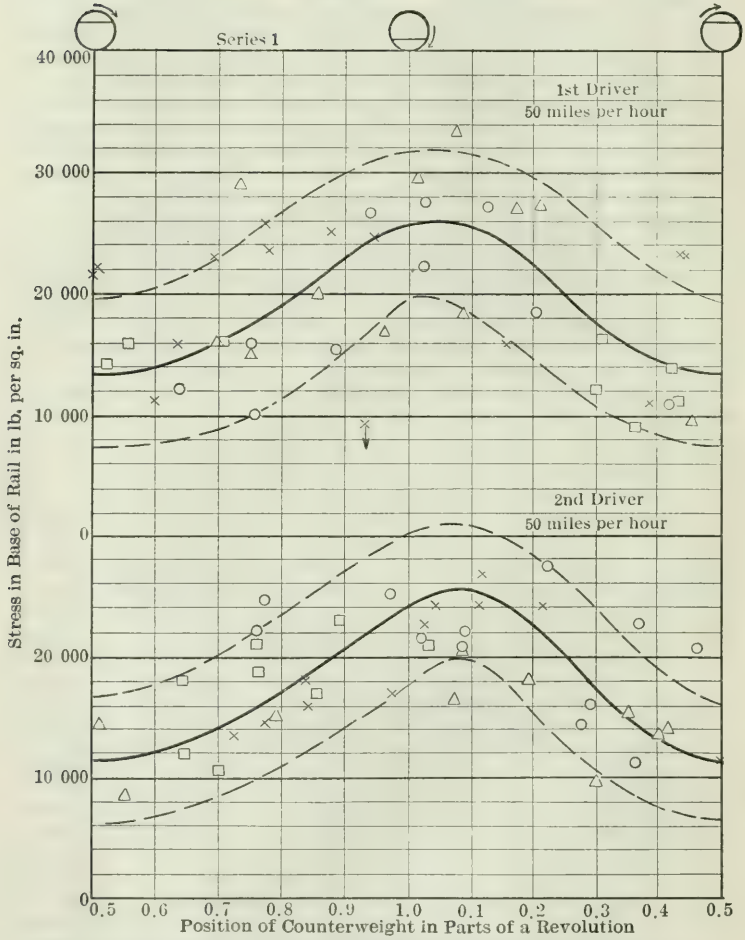


FIG. 10.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

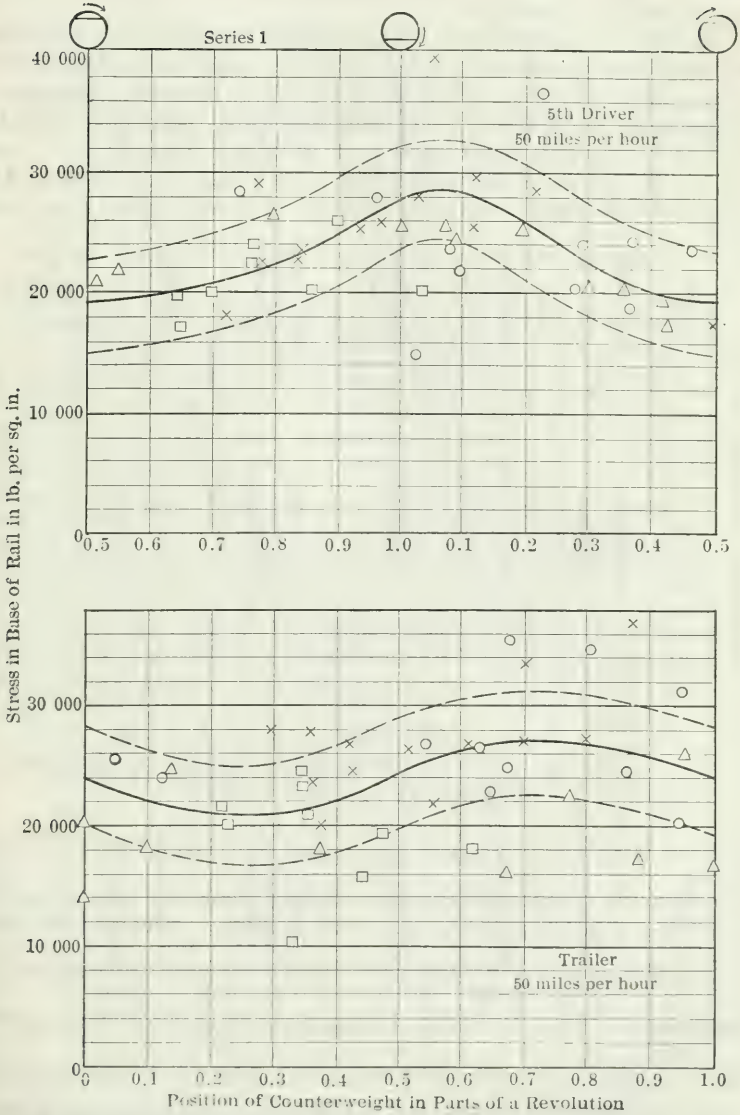


FIG. 11.—OBSERVED VALUES OF STRESS IN RAIL WITH SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

higher speed a noticeable variation exists, the high point of the curve occurring some time after the counterweights of the drivers have passed their high position.

In Table 5 are given the numerical values of the stress in rail under the several wheels as shown in Fig. 13. The mean values given are the mean values taken from the curves of Fig. 12 for an entire revolution.

In Table 6 are given ratios calculated from the data of Table 5. S_5 is the average stress in rail at 5 miles per hour. S_{25}, S_{40}, and S_{50} are the stresses in rail at the high point of curve for 25, 40, and 50 miles per hour, respectively. S_m is the stress found by taking the mean

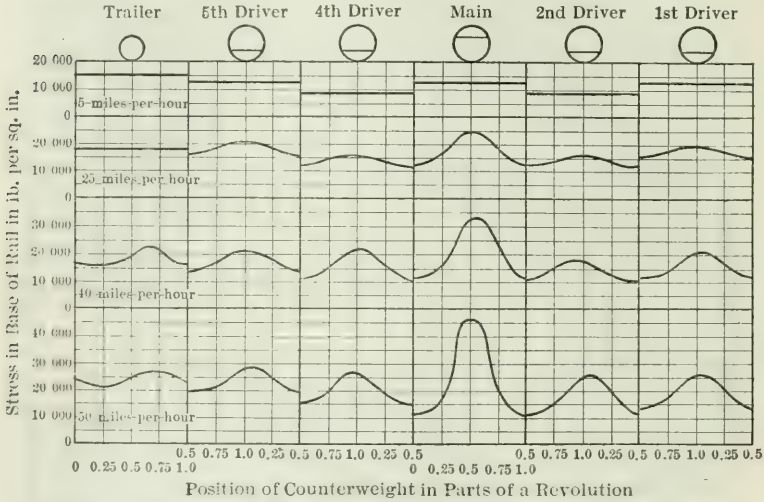


FIG. 12.—CURVES OF AVERAGE STRESS IN RAIL THROUGHOUT REVOLUTION OF DRIVER, SERIES 1, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

ordinate of the curve for the speed used. $\frac{S_{50}}{S_5}$ (called total ratio), may be considered to represent the total effect of change in speed from 5 to 50 miles per hour, including the speed or impact effect and the effect of counterbalance. $\frac{S_m}{S_5}$ (the speed effect ratio) may be considered to represent the effect of speed as distinguished from effect of counterbalance. $\frac{S_{50} - S_m}{S_5}$ (the counterbalance effect) may be considered to represent the effect (increase or decrease) which may be attributed to counterbalance. The division into speed effect and counterbalance effect in this way is quite imperfect, but may be of service in helping to judge of these two effects.

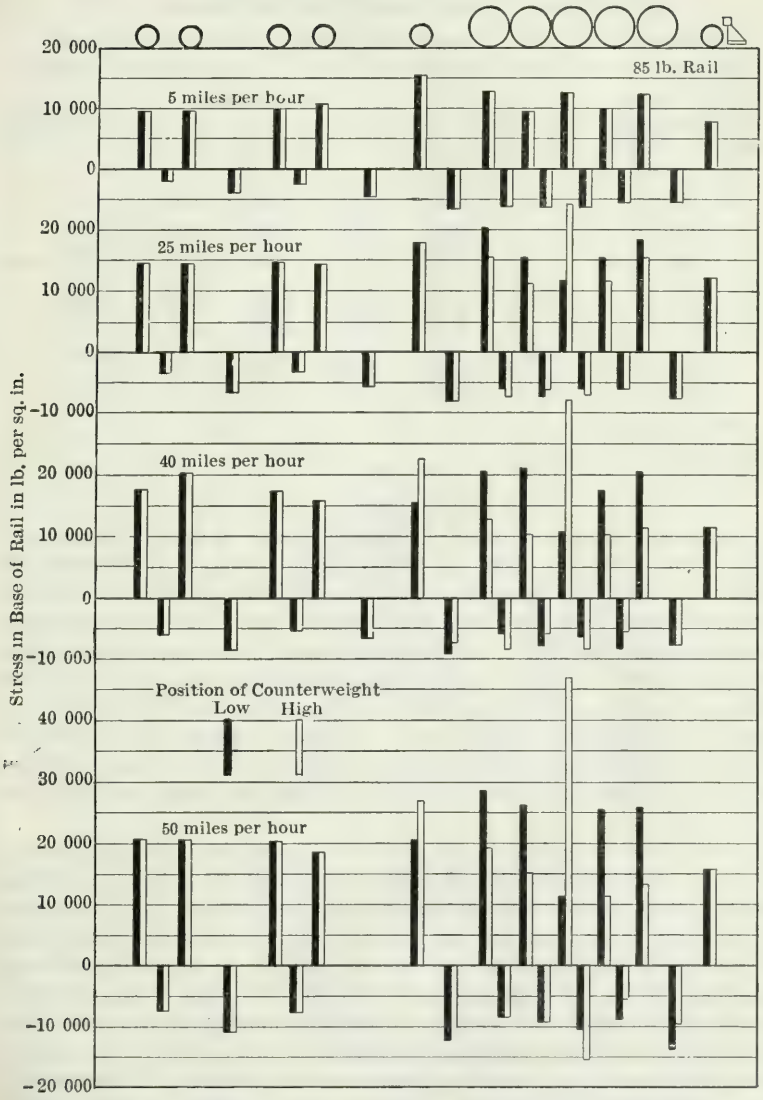


FIG. 13.—STRESS IN RAIL AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 1, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

TABLE 5.—STRESSES IN RAIL WITH SANTA FE TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY—MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 1.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counter-weight.	Trailer.	Driver Number.					Front truck wheel.
			5	4	Main.	2	1	
5	Mean value...	15 200	12 700	9 600	12 500	9 800	12 500	7 900
25	Up.....	.....	15 600	11 400	24 500	12 000	15 600	
	Down.....	.....	20 300	15 600	12 000	15 600	18 700	
	Mean value...	18 000	17 700	14 300	17 300	13 500	17 300	11 900
40	Up.....	22 400	13 000	10 400	32 200	10 400	11 500	
	Down.....	15 600	20 800	21 300	10 900	17 700	20 800	
	Mean value...	18 000	17 200	16 100	18 700	13 500	15 300	11 400
50	Up.....	27 000	19 300	15 100	46 800	11 400	13 500	
	Down.....	20 800	28 600	26 500	11 400	25 500	26 000	
	Mean value...	23 900	23 100	18 900	23 900	17 900	19 100	15 600
Calculated additional stress due to counter-balance at 50 miles per hour.....		600	8 100	11 400	24 200	11 300	8 100	700

Ratios for calculated effect of counterbalance in producing stress in rail are also given in Table 6. These were obtained by dividing the calculated resulting upward or downward pressure effect due to the dynamic augment of the rotating parts at the given speed by the static load pressure, the effect of all adjacent wheels upon moment and stress being considered in determining both static effect and counterbalance effect, as has already been referred to in Article 4, "The Problem of Counterbalancing the Locomotive". To make clearer the method of calculation used in determining the calculated counterbalance ratio, as affecting the moments and stresses in rail, the following examples may be cited.

A reference to the principles given in the analysis of track action in the first progress report (pages 1203 to 1215) will show that the moment and therefore the stress produced by a combination of wheel loads may be found by calculating the load for a single wheel which is equivalent to the combined effect of the several wheels in moment-producing effect, and using the resulting load (which will be termed the equivalent single wheel load) in the formula for bending moment in the rail. This formula for bending moment in the rail then is $M_0 = 0.318\ Px_1$ (Equation (8), page 1205 of the first progress report), where P is the equivalent single wheel load and x_1 is the distance from wheel load to point of zero bending moment in rail for a single wheel load on the track in question.

The calculation of the equivalent single wheel load may then be made by adding algebraically to the load on the wheel itself the effect of the

TABLE 6.—RATIOS OF MAXIMUM STRESS, MEAN STRESS, AND ADDITIONAL STRESS TO STRESS AT 5 MILES PER HOUR, SANTA FE TYPE LOCOMOTIVE, SERIES 1.

Position of counterweight.	Trailer.		Driver Number.										Truck Wheel.	
			5		4		Main.		2		1			
	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.
Average stress at 5 miles per hour, as basis of comparison.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
25 MILES PER HOUR.														
Total effect $\frac{S_{25}}{S_5}$	1.18	1.18	1.23	1.60	1.20	1.63	1.96	0.96	1.22	1.60	1.25	1.54	1.50	1.50
Effect of speed $\frac{S_m}{S_5}$	1.18	1.18	1.42	1.42	1.50	1.50	1.38	1.38	1.38	1.38	1.38	1.38		
Effect of counterbalance (increase or decrease). $\frac{S_{25} - S_m}{S_5}$	0	0	− 0.19	+ 0.18	− 0.30	+ 0.13	+ 0.58	− 0.42	− 0.16	+ 0.22	− 0.13	+ 0.16		
Calculated effect of counterbalance	+ 0.01	− 0.01	− 0.13	+ 0.13	− 0.23	+ 0.23	+ 0.43	− 0.43	− 0.25	+ 0.25	− 0.12	+ 0.12	+ 0.02	− 0.02
40 MILES PER HOUR.														
Total effect S_{40}	1.47	1.03	1.03	1.64	1.09	2.23	2.58	0.88	1.06	1.81	0.92	1.67	1.45	1.45
Effect of speed $\frac{S_m}{S_5}$	1.18	1.18	1.33	1.33	1.69	1.69	1.54	1.54	1.38	1.38	1.23	1.23		
Effect of counterbalance (increase or decrease). $\frac{S_{40} - S_m}{S_5}$	+ 0.29	− 0.15	− 0.30	+ 0.31	− 0.60	+ 0.54	+ 1.04	− 0.66	− 0.32	+ 0.43	− 0.31	+ 0.41		
Calculated effect of counterbalance	+ 0.02	− 0.02	− 0.33	+ 0.33	− 0.59	+ 0.59	+ 1.10	− 1.10	− 0.65	+ 0.65	− 0.31	+ 0.31	+ 0.05	− 0.05
50 MILES PER HOUR.														
Total effect $\frac{S_{50}}{S_5}$	1.78	1.38	1.52	2.25	1.58	2.77	3.75	0.92	1.17	2.60	1.08	2.08	1.97	1.97
Effect of speed $\frac{S_m}{S_5}$	1.58	1.58	1.82	1.82	1.98	1.98	1.92	1.92	1.83	1.83	1.53	1.53		
Effect of counterbalance (increase or decrease). $\frac{S_{50} - S_m}{S_5}$	+ 0.20	− 0.20	− 0.30	+ 0.43	− 0.40	+ 0.79	+ 1.83	− 1.00	− 0.66	+ 0.77	− 0.45	+ 0.55		
Calculated effect of counterbalance	+ 0.03	− 0.03	− 0.51	+ 0.51	− 0.91	+ 0.91	+ 1.74	− 1.74	− 1.01	+ 1.01	− 0.48	+ 0.48	+ 0.08	− 0.08

adjoining wheels, much as was done by means of ratios on page 1210 of the first progress report, since the combined effect is the algebraic sum of the effects of the individual wheel loads. Thus, for the Santa Fe type locomotive, the equivalent single wheel load for finding moments and stresses under the main driver for static loading by this method would be found as follows, using $x_1 = 29.3$ in., and taking values of coefficients corresponding to the effect at various distances found on the master diagram for bending moments given in Fig. 5, page 1207 of the first progress report:

Main driver load			+ 31 600
Effect of first driver	$\frac{x}{x_1} = 4.30$	29 500 × (— 0.03)	— 900
Effect of second driver	$\frac{x}{x_1} = 2.15$	28 100 × (— 0.21)	— 5 900
Effect of fourth driver	$\frac{x}{x_1} = 2.15$	29 600 × (— 0.21)	— 6 200
Effect of fifth driver	$\frac{x}{x_1} = 4.30$	29 200 × (— 0.03)	— 900
Equivalent single wheel load at position of main driver			+ 17 700

The equivalent additional single wheel load (positive or negative) due to counterbalance for a speed of 50 miles per hour, for use in calculating the additional stress in rail due to counterbalance, would be found as follows. For the counterbalance used in the calculations the upward and downward pressures at 50 miles per hour, the dynamic augments of the rotating parts, were calculated to be + 12 300, + 12 400, — 24 900, + 12 400, and + 12 300 lb. for the first, second, third, fourth, and fifth drivers, respectively. The equivalent additional single wheel load at the main driver, which may be attributed to the counterbalance will first be calculated for the counterweight up. For this position underbalance is counted as positive, since it produces additional downward pressure when the counterweight is up, and overbalance on the adjacent driver is counted as negative since it increases the pressure on that wheel and thus lightens the effect at the wheel under consideration. The values of the factors used are taken from Fig. 5 of the first progress report.

Dynamic augment of main driver		+ 24 900
Effect of dynamic augment of first driver	12 300 × 0.03	+ 400
Effect of dynamic augment of second driver	12 400 × 0.21	+ 2 600
Effect of dynamic augment of fourth driver	12 400 × 0.21	+ 2 600
Effect of dynamic augment of fifth driver	12 300 × 0.03	+ 400
Equivalent additional single wheel load at position of main driver		+ 30 900
Equivalent static wheel load		+ 17 700
Total equivalent single wheel load at 50 miles per hour		+ 48 600

Hence, the following ratio is obtained:

$$\frac{\text{Equivalent additional single wheel load at 50 m. p. h.}}{\text{Equivalent static single wheel load}} = \frac{30\,900}{17\,700} = 1.74$$

This ratio represents the moment-making effect of the counterbalance expressed in terms of the moment-making load under static conditions; it may be considered that the stress in rail will increase as this ratio if the assumptions involved in the analysis are not modified by other features.

For counterweight down the effects of the counterbalance for the wheels adjoining the main driver will each be negative instead of positive, and the total equivalent static wheel load is $-30\,900 + 17\,700 = -13\,200$. The ratio for the counterweight down then is $-\frac{30\,700}{17\,700} = -1.74$. The negative moment in rail here indicated should not be taken to mean that there is negative pressure or no pressure on the rail at the main driver, or that the rail is not depressed; the pressure of the main driver on the rail by the analysis used would be 6 700 lb. for the assumed condition. The rail depression would be that due to a single wheel load of 13 500 lb.

For the fourth driver the calculations for static load are:

Fourth driver load		+ 29 600
Effect of second driver $\frac{x}{x_1} = 4.30$	28 100 × (− 0.03)	− 800
Effect of main driver $\frac{x}{x_1} = 2.15$	31 600 × 0.21	− 6 600
Effect of fifth driver $\frac{x}{x_1} = 2.15$	29 200 × 0.21	− 6 100
Equivalent single wheel load at position of fourth driver		+ 16 100

The equivalent additional single wheel load at the fourth driver, which may be attributed to counterbalance, for counterweight up, is found as follows:

Dynamic augment of fourth driver		− 12 400
Effect of dynamic augment of second driver	− 12 400 × (− 0.03)	+ 400
Effect of dynamic augment of main driver	24 900 × (− 0.21)	− 5 200
Effect of dynamic augment of fifth driver	− 12 300 × (− 0.21)	+ 2 600
Equivalent additional single wheel load at position of fourth driver		− 14 600
Equivalent static wheel load		+ 16 100
Total equivalent single wheel load at 50 miles per hour		+ 1 500

The ratio of equivalent additional single wheel load at 50 miles per hour to equivalent static single wheel load (calculated counterbalance ratio) then is $-\frac{14\,600}{16\,100} = -0.91$.

For counterweight down the total equivalent single wheel load becomes + 30 700 and the counterbalance ratio + 0.91.

Values of the calculated effect of counterbalance, obtained by the method described, are given in Table 6 for all the drivers and the trailer and for three speeds. The values in effect give the ratio of the calculated stress in rail due to counterbalance to the stress in rail due to static loading.

The calculated stress in rail due to static loading and to counterbalance may be found, of course, by applying the equivalent single wheel load obtained as above in Equation (8) of the first progress report giving the bending moment in rail, $M_0 = 0.318\ P\ x_1$, and substituting the value of the section modulus of the rail used. Values so computed are given in the tables.

The data of the tests given in the diagrams and tables for the Santa Fe type locomotive show considerable variations in stress with changes in speed. For the position of counterweight which gives maximum effect (counterweight up for the main driver and counterweight down for the other drivers) there is a material increase in stress with increase in speed, particularly at the main driver where the stress at 50 miles per hour is 46 800 lb. per sq. in., a ratio of 3.75 to the stress found at 5 miles per hour. For the opposite position of counterweight the change in stress with increase in speed is generally small, with several cases of decrease in stress.

The ratios given in Table 6 permit some comparison to be made on the effect of speed and of counterbalance taken separately, though, as has been stated, the method of separation is somewhat crude. By this method of separation, the effect of speed alone at 50 miles per hour is to give an additional stress in rail above the stress at a speed of 5 miles per hour which varies from 53 to 98 per cent. The main driver shows the greatest increase due to speed. The increase in stress due to counterbalance for the position which gives maximum effect does not vary greatly from the ratios for calculated effect of counterbalance; for some drivers it is more and for others less. For the main driver the increase in stress at 50 miles per hour, due to speed alone, is 92% of the stress found at 5 miles per hour. For the opposite position of counterweight the stress is generally greater than that which would be expected from calculations of counterbalance effect. It is to be expected that the results will show irregularities and that variations will occur. The deflection of the locomotive springs and the depression of track will change in amount at each wheel with the changing pressure as the position of counterweight changes, and these changes will

influence the pressures in the rail. Such changes may be relatively large at the main driver and adjoining drivers since the lack of balance for adjoining drivers is in opposite directions. The effect of the equalizing levers in transferring load to adjoining wheels complicates the situation and may account for some variations in stress.

For a speed of 40 miles per hour, relations similar to those found at 50 miles per hour exist. The stress found under the main driver is 2.58 times that at a speed of 5 miles per hour and the counterbalance effect itself results in doubling the stress in rail. At 25 miles per hour, the combined effect of speed and counterbalance nearly doubles the stress under the main driver, 58% of this increase apparently being attributable to effect of counterbalance. The ratios for speed and for counterbalance effect at this speed are similar to those found for 50 miles per hour.

It may be noted that had the readings on the right side of the locomotive only been used in this comparison the stresses at the several speeds would have been lower and the ratios in Table 6 would have been higher. Considering the uncertainty concerning the weights of the rotating parts and especially that concerning their exact distribution among the several drivers, it cannot be expected that a closer comparison may be made. The effect of speed at 50 miles per hour is to give an increase in rail stresses, ranging from 50 to 100%, and the effect of counterbalance in producing increased stress is close to that obtained by the method of calculation used.

The calculation for effect of counterbalance at the main driver given on page 1444 indicates that when the counterweight is down, the combined effect of driver loads and counterbalance effect would produce a negative moment in the rail at the main driver, and this negative moment might be expected if the effect of speed did not itself contribute to a positive moment at the wheel. This effect of speed is so large that in all the tests the stress at the main driver with counterweight down indicates the presence always of a positive bending moment. The foregoing relates to stress and bending moment—the calculated rail pressure and rail depression have to be treated separately from stress and moment; even without the effect of speed there is no indication from the calculations given that the wheel would leave the rail.

Attention is called to the variation in stress at the trailer for the higher speeds. There is an appreciable change in stress as the counterweights of the drivers change position. The maximum stress occurs somewhat after the counterweight has reached its high position. Due to the large load on the trailer, its distance from other wheels, and the effect of counterbalance, the stress at this wheel is high, the highest of any except that at the main driver and except for the fifth driver at 5 miles per hour. The data of Series 2 will be found to give similar results. The time at which the maximum stress is found ranges from

the counterweight at six-tenths of a revolution after the counterweight is down to eight-tenths of a revolution; it occurs, therefore, between the positions of counterweight which give the minimum and the maximum stress at the driver ahead of the trailer.

It will be remembered that the values of stresses in rail considered in the foregoing discussion were taken from the curves of averages and that the band which enclosed the greater number of the observations extends 3 000 or 5 000 lb. per sq. in. above the curve and a similar distance below it. Above and below this band are other observed points, and the average of these outlying points is still 3 000 lb. per sq. in. above and below the limit of the bands. It may be expected that values may frequently be developed which are 3 000 or 5 000 lb. per sq. in. higher or lower than those used in the discussion and that not infrequently values of 3 000 lb. per sq. in. still greater or less will be found. Values 6 000 to 8 000 lb. per sq. in. greater than the stresses given in the discussion therefore may be expected not infrequently.

11.—The Santa Fe Type Locomotive; Rotating Parts Lightened.— In an effort to reduce the injurious effect of the Santa Fe type locomotives upon the track, the Railroad Company lightened certain of the rotating and reciprocating parts of Locomotive No. 19 by cutting away metal at various places. A series of runs were made with this locomotive (Series 2) and comparisons will be made with the results of Series 1 which have been described in the preceding article. In making a comparison it should be borne in mind that the results found with two locomotives of nominally the same construction may differ considerably. The tests of Series 1, with locomotives as received from the builders, were made with two locomotives, Nos. 22 and 28. The tests of Series 2 were made at the same place in the track, with the instruments distributed in the same way, and the conditions of the test were much the same. The readings from the instrument on the left rail were included in the data used, as was the case for Series 1.

The statement from the office of the General Superintendent of Motive Power shows reductions in weight of rotating parts as follows: back end of main rod (considered to be tributary to the main driver in calculating its dynamic augment), 42 lb.; front end of side rod (tributary to first driver), no change; front intermediate side rod (tributary to second driver), 20 lb.; middle connecting side rod (tributary to main driver), 75 lb.; back intermediate side rod (tributary to fourth driver), 20 lb.; back end side rod (tributary to fifth driver), no change. Eighty-six pounds of metal were taken from the piston; 52 lb. from the cross-head, and 34 lb. from the front end of the main rod, but whatever the effect of the lightening of these reciprocating parts on the locomotive itself and on the effect of speed upon the track, this reduction does not alter the effect of rotating parts.

In computing the effect of counterbalance for Locomotive No. 19, the weights of rotating parts given in Table 1, page 1423, were reduced by the amounts previously given. The equivalent counterbalance at the pin of the main driver is 629 lb. instead of 746 lb. and that of the second and fourth driver 390 lb. instead of 370 lb. It will be seen that the underbalance of the main driver is decreased and the overbalance (for rotating parts) of the other two drivers named is increased, the first and fifth drivers not having any change. The effect of the increase of overbalance of the second and fourth drivers upon the stress under these two drivers is nearly neutralized by the influence of the decrease in underbalance of the main driver, and calculations indicate that the resulting effect at the second and fourth drivers is negligible. The cal-

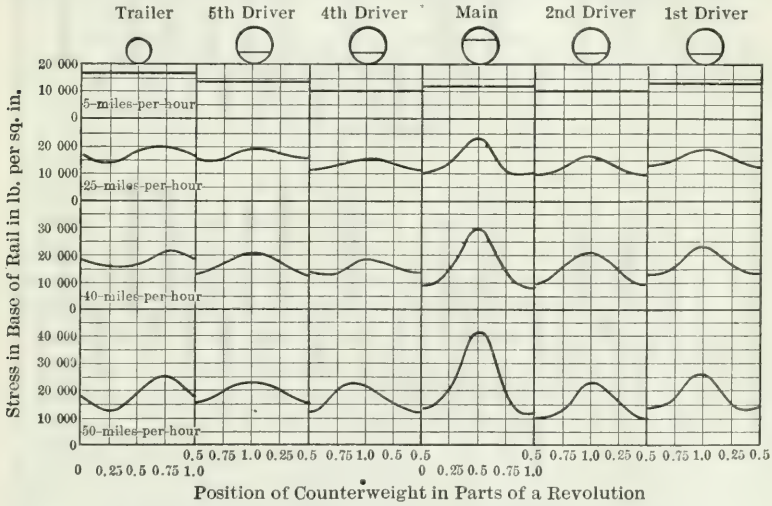


FIG. 14.—CURVES OF AVERAGE STRESS IN RAIL THROUGHOUT REVOLUTION OF DRIVER, SERIES 2, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

culated effect at the main driver however will be to decrease the stress in rail due to dynamic augment about 11 per cent. The calculated decrease in stress in rail at the main driver for these conditions would be 1 800 lb. per sq. in. at 40 miles per hour and 2 800 lb. per sq. in. at 50 miles per hour.

Fig. 14 gives the stress in base of rail throughout a revolution of driver for all the drivers and the trailer for Series 2, obtained in the same way as were those already given for Series 1, and Fig. 15 gives the value of the stresses in base of rail at the various wheels for the high point and the low point of these stress curves. Table 7 gives numerical values of these stresses, as well as the values of the mean stress for the several stress curves. In Table 8 are given ratios which

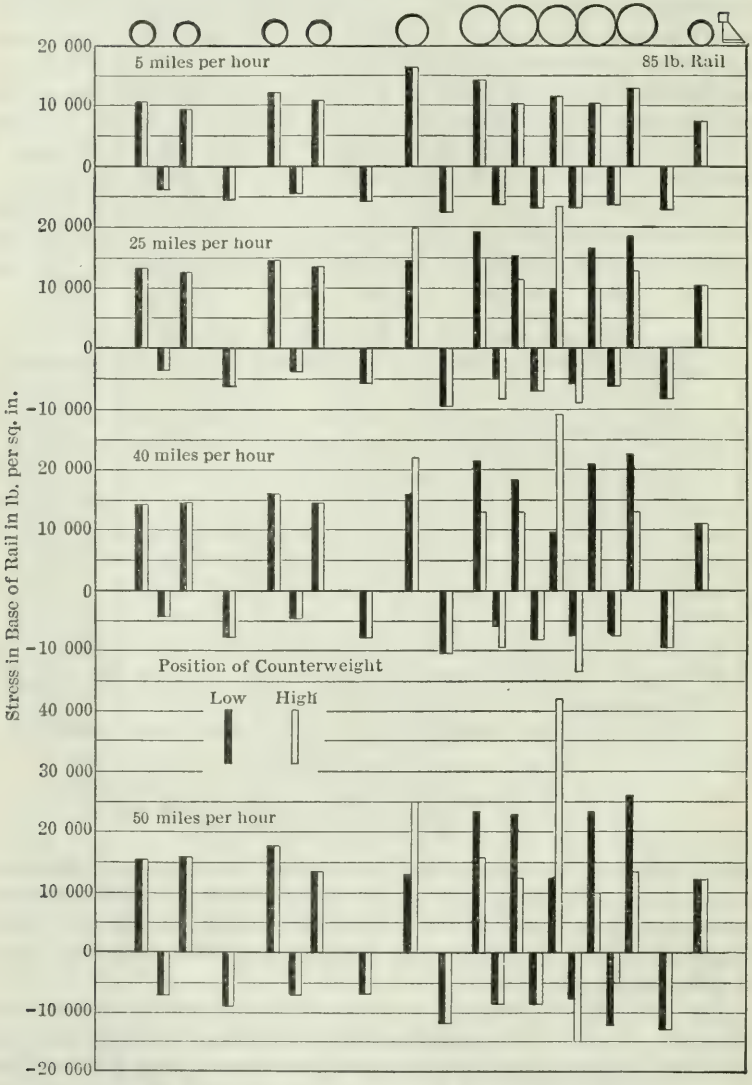


FIG. 15.—STRESS IN RAIL AT HIGH AND LOW POSITION OF COUNTERWEIGHT. SERIES 2, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

TABLE 7.—STRESSES IN RAIL WITH SANTA FE TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY—MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 2, PARTS LIGHTENED.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.					Front truck wheel.
			5	4	Main.	2	1	
5	Mean value..	16 300	14 400	10 400	11 400	10 400	13 000	7 600
25	Up.....	19 800	14 900	11 400	23 400	10 000	13 000	
	Down.....	14 600	19 200	15 100	9 400	16 600	18 700	
	Mean value..	16 300	16 800	13 400	14 600	12 800	15 800	10 200
40	Up.....	21 800	13 000	13 000	29 100	10 000	13 000	
	Down.....	15 600	21 300	18 200	9 400	20 800	23 400	
	Mean value..	17 800	17 500	15 500	17 100	15 100	17 200	10 800
50	Up.....	25 000	15 600	12 500	42 000	10 000	13 500	
	Down.....	13 000	23 400	22 900	12 500	23 400	26 000	
	Mean value..	18 500	20 200	17 500	23 900	15 400	18 200	12 200
Calculated additional stress due to counter balance at 50 miles per hour.....		600	7 900	11 300	21 100	11 200	7 900	700

may be considered to express the effect of speed and the effect of counterbalance. These correspond to the ratios given in Table 6. Ratios expressing the calculated relative effect of counterbalance are also given.

Comparisons of the two series to determine effect of counterbalance have less value because the locomotives used in the two series give somewhat different speed effects and different distributions of stress under the several drivers, the increase in mean stress over that at 5 miles per hour being generally smaller in Series 2 than in Series 1. The advantage obtained through lightening the rotating parts may be shown by comparing the stress at the main driver with the average of the stresses at the adjacent drivers, taking into account also the differences in stress which exist at 5 miles per hour for the two series, differences which may be due to a different distribution of load among the drivers in the locomotives. For both counterweight up and counterweight down the difference between the stress at the main driver and the average of the stresses at the adjacent drivers is substantially less for Locomotive No. 19 than for the other two locomotives; this is true at speeds of 40 and 50 miles per hour to an amount beyond the difference in stress at these drivers for 5 miles per hour.

The effect of the lightening of rotating parts is seen also by comparing the differences between maximum stress and mean stress for the main driver and adjoining drivers for Series 1 and 2 (Tables 5 and 7); the effect of counterbalance is less for the main driver in Series 2 than in Series 1 as compared with the change for the average of the two adja-

TABLE 8.—Ratios of Maximum Stress, Mean Stress, and Additional Stress to Stress at 5 Miles per Hour, Santa Fe Type Locomotive of the St. Louis-San Francisco Railway, Series 2, Parts Lightened.

Position of counterweight.	Trailer.		Driver Number.										Truck wheel.	
			5		4		Main.		2		1			
	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.
Average stress at 5 miles per hour assumed as basis of comparison..	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
25 Miles per Hour.														
Total effect..... $\frac{S_{25}}{S_5}$	1.21	0.89	1.04	1.34	1.10	1.45	2.04	0.82	0.95	1.60	1.00	1.44	1.34	1.34
Effect of speed..... $\frac{Sm}{S_5}$	1.00	1.00	1.17	1.17	1.29	1.29	1.27	1.27	1.23	1.23	1.22	1.22		
Effect of counterbalance (increase or decrease)... $\frac{S_{25} - Sm}{S_5}$	+ 0.21	− 0.11	− 0.13	+ 0.17	− 0.19	+ 0.16	+ 0.77	− 0.45	− 0.28	+ 0.37	− 0.22	+ 0.22		
Calculated effect of counterbalance.	+ 0.01	− 0.01	− 0.12	+ 0.12	− 0.23	+ 0.23	+ 0.38	− 0.38	− 0.25	+ 0.25	− 0.12	+ 0.12	+ 0.02	− 0.02
40 Miles per Hour.														
Total effect..... $\frac{S_{40}}{S_5}$	1.34	0.96	0.91	1.49	1.25	1.75	2.55	0.82	0.95	2.00	1.00	1.80	1.43	1.43
Effect of speed..... $\frac{Sm}{S_5}$	1.09	1.09	1.22	1.22	1.49	1.49	1.49	1.49	1.45	1.45	1.32	1.32		
Effect of counterbalance (increase or decrease).. $\frac{S_{40} - Sm}{S_5}$	+ 0.25	− 0.13	− 0.31	+ 0.27	− 0.24	+ 0.26	+ 1.06	− 0.67	− 0.50	+ 0.55	− 0.32	+ 0.48		
Calculated effect of counterbalance.	+ 0.02	− 0.02	− 0.32	+ 0.32	− 0.58	+ 0.58	+ 0.97	− 0.97	− 0.64	+ 0.64	− 0.30	+ 0.30	+ 0.05	− 0.05
50 Miles per Hour.														
Total effect..... $\frac{S_{50}}{S_5}$	1.53	0.79	1.09	1.63	1.20	2.20	3.68	1.09	0.95	2.25	1.04	2.00	1.61	1.61
Effect of speed..... $\frac{Sm}{S_5}$	1.13	1.13	1.41	1.41	1.68	1.68	2.09	2.09	1.48	1.48	1.40	1.40		
Effect of counterbalance (increase or decrease).. $\frac{S_{50} - Sm}{S_5}$	+ 0.40	− 0.34	− 0.32	+ 0.22	− 0.48	+ 0.52	+ 1.59	− 1.00	− 0.53	+ 0.77	− 0.36	+ 0.60		
Calculated effect of counterbalance.	+ 0.03	− 0.03	− 0.50	+ 0.50	− 0.90	+ 0.90	+ 1.52	− 1.52	− 1.00	+ 1.00	− 0.47	+ 0.47	+ 0.08	− 0.08

cent drivers. It seems probable that the lower average increase in stress due to speed in Series 2 may also be due, in part, to the lightening of parts.

A marked difference in the two series, however, lies in the greater effect of speed and counterbalance for 50 miles per hour found in Series 1 as compared with Series 2. Much of this difference is in speed effect alone. If the stresses at drivers and trailer be summed for the two series it will be seen that for 50 miles per hour the sums of maximum stress, minimum stress, mean stress, stress with counterweight up, and stress with counterweight down will each be about 10% of the mean stress more for Series 1 than for Series 2, while for 25 and 40 miles per hour there is relatively little difference. The same effect may be seen by comparing the ratios of Tables 6 and 8.

As has already been stated, in Series 1, Locomotive No. 22 was used in the runs at 5 and 40 miles per hour and part of the runs at 25 miles per hour, and Locomotive No. 28 was used in the remaining runs at 25 miles per hour and in all the runs at 50 miles per hour. The results for the tests at 25 miles per hour have not been separated for the two locomotives used in Series 1. The results show that locomotives of the same nominal construction may have quite different distribution of loads among the drivers.

The stresses in rail for the locomotive with rotating parts lightened are still very high at speeds of 25 miles per hour or more.

12.—Comparison of Stress Under the Two Sides of the Santa Fe Type Locomotive.—The experience of the St. Louis-San Francisco Railway with these Santa Fe type locomotives gave some indication that injuries were more common or greater in the rail at the left side of the locomotive than in the other rail. To learn whether the stresses in rail were the same under the two sides of the locomotive, tests were made with Locomotive No. 22. The four instruments were placed on one rail as shown in Fig. 4, and runs were made in one direction at speeds of 5, 25, and 40 miles per hour, forming Series 3, the instruments being on the rail at right side of locomotive. The locomotive was then reversed in direction and runs over the same track were made in this direction at the same speeds, forming Series 4 for instruments on rail at left side of locomotive. The instruments retaining their position on the same rail and other conditions remaining the same, and all the tests being made on one day, results were thus obtained for the effect of the two sides of the locomotive which seem comparable.

The data were reduced in the usual way and plotted according to position of counterweight, and the maximum value of the stress curve for each wheel at each speed was determined. It was found that the stress in rail under the left side of this locomotive was considerably greater than that under the right side. That this difference is not

caused by speed or counterbalance is indicated by a marked difference at a speed of 5 miles per hour; the sum of the stresses in rail at the five drivers, trailer, and truck wheel, as found for the left side of the locomotive, is as much as 20% greater than the sum of the corresponding stresses for the right side of the locomotive. A similar difference is found at all speeds, both for mean stresses and for maximum stresses. It seems that the difference is due to a cause not related to speed.

That this excess of stress at one side of the locomotive was not due to methods of testing is shown by the stresses at the wheels under the tender. The average stress for the three speeds for Series 3 (rail at right side of locomotive) is 15 500 lb. per sq. in. and for Series 4 (rail at left side of locomotive) is 15 300 lb. per sq. in. A few runs of Series 3 were made before the noon hour; the remaining runs and all of Series 4 were made after the noon hour. There would be some decrease in the load of coal and water, but this would not be large.

In all the other tests with the Santa Fe type locomotive, one instrument was placed on the rail at the left side of the locomotive opposite the middle one of the three instruments on the rail at the right side, as shown in Fig. 4. In the plots of data, it was found that the results obtained with the instrument at the left side generally average higher than the results obtained with the other instruments, thus agreeing in this respect with the series just discussed. Whatever the cause of this variation, it has made complications in the interpretation of the data for other purposes and has thrown uncertainty on some comparisons which it was desired to make, but the relatively small amount of data available has made it seem best generally to include all the instrumental results even with this added complication. The use of one instrument on the second rail has seemed advisable for the purpose of checking up in possible cases of irregular or unexpected values in the data.

The reason for this difference in stress developed on the two sides of the locomotive is not known; it is planned to make a further investigation of the conditions.

It should be noted that the lateral bending of rail due to nosing is not here discussed; it may have an important relation to rail injury.

13.—The Effect of Santa Fe Type Locomotive Upon Rail and Track.—The tests made with the locomotive of the Santa Fe type gave an interesting opportunity to find the effect upon rail stresses produced when the desired amount of counterweight is not or can not be placed on the main driver, but instead the deficiency for the given wheel is distributed among the other drivers which thereby have their counterweight increased above what is already an overbalance so far as rotating parts and vertical pressure are concerned. The increased stress in the rail is due not alone to the increased pressure on the rail given by the excess of weight of rotating parts on the main driver when the crank pin is

down, but it may be seen from the illustrative calculations given in Article 10 on "Effect of Speed and Counterbalance" that the overbalance in the adjacent drivers at the same time relieves part of the pressure on the rail at those drivers and the effect of this decrease in pressure is to increase the bending moment in the rail under the main driver.

Similarly, the addition of counterweight to drivers other than the main driver not only increases the counterweight effect in the rail at these drivers (which are already overbalanced so far as rotating parts are concerned), but the deficiency in the counterweight of the main driver decreases the pressure on the rail at the main driver when the counterweight is down and thus increases the moment in the rail at the adjacent drivers. The existence of these influences is evident from the analysis and illustrations already given. Whatever may be the effect of this abnormal distribution of counterweight upon the frame of the locomotive or upon its riding qualities, it is evident that a deviation from a uniformity of counterbalance effect among the several drivers aggravates conditions producing stresses in rail which otherwise may be bad enough.

The Santa Fe type locomotives used in the tests may seem extreme in the amount and distribution of counterweight, but the underbalance of the main driver in locomotives of this type as used on some railroads is reported to be considerably greater than that found on these locomotives; on the other hand, the Santa Fe type locomotive of at least one railroad has an underbalance of less than half that of those on the St. Louis-San Francisco Railway. In other features of the design also these locomotives were not calculated to produce low or even stresses in the rail. The weight given for the main driver, 31 600 lb., is considerably greater than that on the second and fourth drivers, which might advantageously be given more load than the first and fifth. Even at low speeds the second and fourth drivers give considerably lower stresses than the others. Besides, the effect of the cylinder pressure at mid-stroke when the locomotive is working is to give a vertical component at the crank pin which serves to increase the pressure on the rail under the main driver both for counterweight up and counterweight down. The weight on the trailer, a wheel located some distance from other wheels, and thus having little moment-decreasing influence from the other wheels in these locomotives, is too great to produce even effects in rail stresses, the stress in rail at this wheel at low speeds (see Table 5) being considerably greater than those at the drivers, and at higher speeds generally being the highest at any point except at the main driver.

The stresses in rail produced by these locomotives at speeds of 25 miles per hour are high. It should be borne in mind, too, that the values under discussion are the average stresses for the runs, and that the occa-

sional maximum stress is much higher, and that the stress at one side of the rail will be still higher. Speed and counterbalance were found to increase stresses very rapidly. For the main driver the ratios of the stresses at the higher speeds to the stress at 5 miles per hour (2.04 for 25 miles per hour, 2.54 for 40 miles per hour, and 3.68 for 50 miles per hour) show undesirable and unsatisfactory conditions in a locomotive in so far as the effect on the maintenance and life of the track is concerned. The occurrence of stresses in base of rail (the average of the values at the two edges) as high as 52 000 lb. per sq. in. (and 10% higher in head of rail), observed at high speeds in a number of cases where the records were well defined, shows that it is not strange that kinks and rail failures were produced in the regular service of these locomotives. In some of the runs at 50 miles per hour, kinks in rail were developed near the rail on which instruments were placed. Even at lower speeds than those which may have produced such visible injury to track, the punishment of the track and the effect of continued application of such stresses probably would be sufficient to cause considerable injury to the track structure. Such locomotives should run only at low speeds, and every precaution should be taken to prevent even the occasional use of higher speeds on heavy down grades in undulating country or through other carelessness. The observance of much higher stresses under the left side of the locomotive than under the right it is hoped will lead to investigation to determine the origin of this undesirable effect upon track.

14.—Tests with the Pacific Type Locomotive.—The main purpose of the tests with the Pacific type of passenger locomotive made on the St. Louis-San Francisco Railway was to find the increase in stress in rail due to speed and counterbalance. The Railroad Company, however, desired to learn the effect of certain variations in these locomotives—the addition of a mechanical stoker, different arrangements of an equalizing lever, and the lightening of rotating parts—and the tests were outlined with a view of including the effect of these variations in the locomotives. Part of the changes in locomotive were relatively small and their extent had not been realized when the tests were started; the resulting change in distribution of load among the several wheels was also uncertain. The demands for locomotives under the existing traffic conditions and the beginning of winter weather made it impracticable to extend the tests to secure an adequate number of runs for some of the purposes of the test. In getting the effect of speed and counterbalance, it has been found best to group series of tests which have different locomotives and different conditions of locomotive, so that some of the effects may be masked by the differing conditions

and the real increases due to speed and counterbalance therefore may be greater than the values reported indicate.

Table 4 gives the variations in the Pacific type locomotives used. Series 8 to 12 were made with the locomotives as received from the builders except as the position of the pin in the equalizing lever was changed for some of the tests. The second location of test (rail adjoining that used for the first test) was used to determine whether the first test location was in any way abnormal; a comparison of the results indicate that there was little difference in conditions.

Series 5 and 6 made with Locomotive No. 1061 having a mechanical stoker added were made at the beginning of the test work on the St. Louis-San Francisco Railway. It is evident from the data that the instruments were not working satisfactorily or the observers were not in good practice. Many records were so indefinite as not to be usable. As the number of runs was small, the number of definite values obtained was too small to use in making comparisons, and these two series will not be reported. As the stoker weighed only 3 500 lb. and the weight added to one wheel was probably much less than 1 000 lb., a large number of runs would have been necessary to secure an adequate amount of data for making a proper comparison.

The Pacific type locomotives were received and generally used with the pin in the rear hole of the equalizer bar at the rear of the locomotive. The change of the pin to the middle hole releases weight from the trailer and increases the weight on the drivers. It is also expected to decrease the weight on the front trucks. The changes in the weights on the several wheels as calculated by the Mechanical Department of the railway may be seen from the following weights:

	Trailer.	Third driver.	Main driver.	First driver.	Two truck wheels.
Pin in rear hole	32 150	29 700	32 250	29 750	28 850
Pin in middle hole..	28 850	31 950	34 450	31 950	25 550

These calculated values indicate that by the change in length of equalizing bars, 3 300 lb. is released from the trailer and 3 300 lb. from the two truck wheels and that this 6 600 lb. is divided equally among the three drivers. Based on the calculated or assumed distribution of the weight of the locomotive among the several wheels, the stress in the rail calculated by the method of combination of wheel loads given in the first progress report would be decreased by 2 300 lb. per sq. in. under the trailer and 1 000 and 1 400 lb. per sq. in., respectively, under the two truck wheels, while the stresses under the three drivers would be increased by 1 800, 1 000, and 1 500 lb. per sq. in., respectively. The test results are not fully comparable for this purpose, and the results of the series with the pin in the two positions are not entirely concordant. The average of the results of the several series indicates that changing

the pin to the middle hole decreases the stress in rail under the trailer, and increases the stress under the three drivers, with little change under the truck wheels. The amount of this change, however, as given by the averages of Series 7 to 12, is much less than the change found by calculation on the basis of the assumed distributions of weight. The effect of speed was noticeably less at the trailer with the pin in the middle hole.

Series 9, 10, 11, and 12 may be grouped together for finding the effect of speed and counterbalance, though the differences in conditions for the four series make comparisons in some ways somewhat uncertain. As there were about the same number of observations for each position of pin in the equalizing bar, this variation should not affect the results materially. The effect of differences in locomotives and of differences in track in the two locations of course is not known, but it is thought to be not large.

The observed values of stresses in base of rail for each speed in each series were plotted with respect to position of counterweight, the values of the plotted points in groups for each twentieth of a revolution were then averaged and a curve drawn to represent as well as possible the position and trend of these points, as has already been described for the tests with the Santa Fe type locomotive. The data for each speed of the four series (9, 10, 11, and 12) were also plotted together and curves drawn which represent the averages of the four series. Fig. 16 gives the resulting curves for the combined series. The maximum, minimum, and mean values for the curves were taken from the diagrams. The values of the stress in rail for the four series and the combined series are given in Table 9. It should be noted that, as was the case in the tests with the Santa Fe type locomotive, the greater number of the plotted observed values was found in a belt on each side of the curve of averages in the diagrams of data, the limits of the belt being 3 000 to 5 000 lb. per sq. in. above and below the curves, while still other points were above and below the belt. Fig. 17 shows graphically the principal stresses of the combined four series, including the average negative stress at points between wheels.

In Table 10 are given the ratios calculated from the data of Table 9 for the combined four series. S_5, S_{45}, and S_{60} are the stresses in rail at speeds of 5, 45, and 60 miles per hour, respectively. S_m is the mean value of the stress curve for the speed used. The total ratio, speed effect ratio, and counterbalance ratio are used as in the discussion of the counterweight with the Santa Fe type locomotive. Calculated counterbalance ratios, obtained with the data of counterbalance given in Article 5 on "The Locomotives", by the method already used, are also given in Table 10. The calculated upward and downward pressures at 60 miles per hour are 12 800, 10 000, and 13 000 lb. for the first, second, and third drivers, respectively. The downward and upward pressures

TABLE 9.—Stresses in Rail with Pacific Type Locomotive of the St. Louis-San Francisco Railway. Maximum, Minimum, and Mean Values from Curves for Series 9, 10, 11, and 12.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.			Truck Wheel.	
			3	Main.	1	2	1
		Series 9					
5	Mean value	20 300	16 200	15 200	15 400	6 000	9 000
45	Up	32 200	21 300	24 400	15 600		
	Down	23 500	27 600	14 600	25 000		
	Mean value	27 700	23 700	19 200	20 800	10 400	13 600
60	Up	33 300	14 100	26 000	15 600		
	Down	22 800	31 200	16 200	26 000		
	Mean value	27 400	21 800	22 200	20 300	12 400	16 000
		Series 10					
5	Mean value	20 890	17 000	15 800	14 100	5 200	8 900
45	Up	33 800	20 000	25 000	17 700		
	Down	29 700	27 600	19 300	24 000		
	Mean value	31 400	23 100	21 300	19 900	14 000	15 400
60	Up	34 800	17 000	24 800	14 100		
	Down	28 100	28 300	15 100	26 800		
	Mean value	30 300	21 900	18 900	19 300	14 700	16 200
		Series 11					
5	Mean value	22 200	17 600	18 000	15 200	5 800	8 600
45	Up	28 100	13 000	28 100	14 000		
	Down	23 400	24 400	14 000	21 300		
	Mean value	25 300	18 200	20 700	16 100	13 000	15 500
60	Up	36 400	18 200	36 400	16 600		
	Down	28 600	34 800	19 800	26 000		
	Mean value	32 100	25 800	25 400	19 700	14 000	18 700
		Series 12					
5	Mean value	23 900	17 200	15 600	14 800	5 600	9 800
45	Up	31 200	18 200	24 100	15 400		
	Down	20 800	29 100	15 600	23 900		
	Mean value	24 200	22 500	19 900	19 600	12 100	15 900
60	Up	.....	19 800	28 600	14 800		
	Down	.....	37 000	20 600	23 900		
	Mean value	35 400	27 500	24 200	21 100	15 200	17 900
		Series 9, 10, 11, and 12—Combined					
5	Mean value	21 800	17 500	16 800	14 700	5 600	9 300
45	Up	31 200	19 800	25 000	15 100		
	Down	25 000	26 500	15 600	22 900		
	Mean value	27 900	21 900	20 200	19 200	12 500	15 100
60	Up	37 000	17 200	28 600	16 100		
	Down	29 600	32 800	17 200	26 000		
	Mean value	32 900	25 100	22 300	20 700	13 900	17 200
Calculated stress under static load		23 500	17 300	17 100	16 400	4 400	9 100
Calculated additional stress due to counter-balance at 60 miles per hour		700	8 700	4 400	8 500	2 100	100

TABLE 10.—Ratios of Maximum Stress, Mean Stress, and Additional Stress to Stress at 5 Miles per Hour, Pacific Type Locomotive. Series 9, 10, 11, and 12 Combined.

Position of counterweight.	Trailer.		Driver Number.						Truck Wheel.			
			3		Main.		1		2		1	
	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.
Average stress at 5 miles per hour assumed as basis of comparison	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
45 Miles per Hour.												
Total effect $\frac{S_{45}}{S_5}$	1.43	1.15	1.13	1.52	1.48	0.93	1.03	1.56	2.21	2.21	1.63	1.63
Effect of speed $\frac{Sm}{S_5}$	1.28	1.28	1.26	1.26	1.20	1.20	1.31	1.31				
Effect of counterbalance (increase or decrease) $\frac{S_{45} - Sm}{S_5}$	+ 0.15	− 0.13	− 0.13	+ 0.26	+ 0.28	− 0.27	− 0.28	+ 0.25				
Calculated effect of counterbalance	+ 0.01	− 0.01	− 0.28	+ 0.28	− 0.15	+ 0.15	− 0.29	+ 0.29	+ 0.27	− 0.27	+ 0.01	− 0.01
60 Miles per Hour.												
Total effect $\frac{S_{60}}{S_5}$	1.69	1.36	0.98	1.88	1.70	1.24	1.10	1.77	2.48	2.48	1.85	1.85
Effect of speed $\frac{Sm}{S_5}$	1.50	1.50	1.44	1.44	1.32	1.32	1.41	1.41				
Effect of counterbalance (increase or decrease) $\frac{S_{60} - Sm}{S_5}$	+ 0.19	− 0.15	− 0.45	+ 0.44	+ 0.38	− 0.30	− 0.31	+ 0.36				
Calculated effect of counterbalance	+ 0.03	− 0.03	− 0.50	+ 0.50	− 0.26	+ 0.26	− 0.52	+ 0.52	+ 0.48	− 0.48	+ 0.01	− 0.01

were considered as positive and negative loads at the several drivers, and the additional equivalent single wheel load for this combination of added loads was calculated for each wheel by the method described in the case of the Santa Fe type locomotive. The ratios of the resulting values to the equivalent single wheel loads found for the combination of nominal static wheel loads was calculated, and is given as the calculated counterbalance ratio.

The ratios in Table 10 may be considered to separate the effect of counterbalance and the effect of speed alone, though it must not be expected that the division of effect will be accurate.

It is seen that the maximum stress in rail at the first and third driver occurs when the counterweight is down at speeds of both 45 and 60 miles per hour, while the maximum stress at the main driver

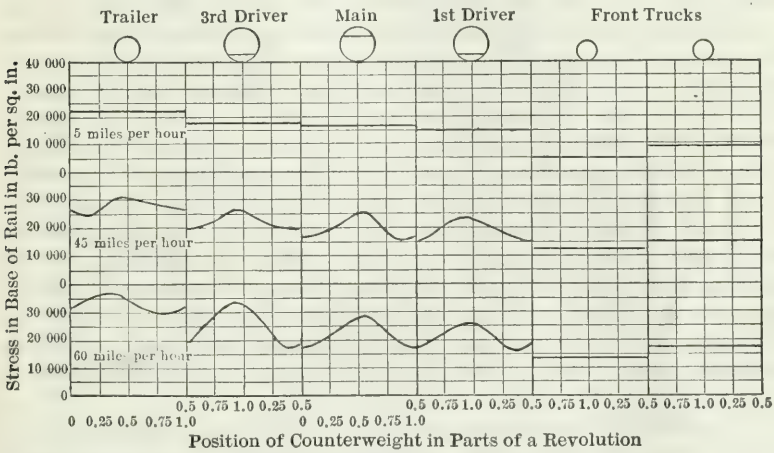

Trailer
3rd Driver
Main
1st Driver
Front Trucks
Stress in Base of Rail in lb. per sq. in.
40 000
30 000
20 000
10 000
0
5 miles per hour
45 miles per hour
60 miles per hour
0 0.25 0.5 0.75 1.0 0.25 0.5 0.75 1.0 0.25 0.5 0.75 1.0 0.25 0.5 0.75 1.0 0.25 0.5
Position of Counterweight in Parts of a Revolution

FIG. 16.—CURVES OF AVERAGE STRESS IN RAIL THROUGHOUT REVOLUTION OF DRIVER, SERIES 9, 10, 11, AND 12 COMBINED, PACIFIC TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

occurs when the counterweight is up. The maximum stress at the trailer occurs when the counterweight of the drivers is up, the exact position being when the drivers have turned about four-tenths of a revolution from the down position of the counterweight. The signs of the counterbalance ratio for the main driver given in Table 10, it is seen, are opposite to the signs of the observed counterbalance ratio for the main driver, while for the other wheels the signs of the calculated and the observed ratios agree.

No explanation is offered except that the weights of the rotating parts may be different from those given in Table 2, or that the assumed effect of such parts as the main rod is in error. The effect of the main rod and side rod being outside the plane of the driver seems to account for only part of the discrepancy. The results are so pronounced and

the information on the position of counterweight for the observations so definite and so general that no doubt can be thrown on the character of the test data in this respect. It may be added that it would not require a large discrepancy in the amount and position of the counterweight of the main driver, or in the weight of the rotating parts tributary to this driver, to produce quite a change in the calculated vertical pressure at the down position of the counterweight, taking into account the effect of the dynamic augment of the adjacent drivers upon the vertical pressure at the main driver. Possibly the weight of rotating parts given may not have been properly divided among the three drivers.

It will be noted that the counterbalance effect upon the trailer is considerably larger than would be expected from the effect due to relief of load on rail at third driver when the counterweight is up; it may receive a portion of the effect of counterbalance of the drivers in some way through the equalizing levers. The maximum stress at the trailer occurs when the counterweights of the drivers have moved about four-tenths of a revolution beyond their low position. It will be observed that the ratios for the counterbalance effect of the drivers is considerable, amounting to 44% of the low speed stress for the third driver at a speed of 60 miles per hour. The method of combining the results of the series may be expected to make the apparent effect of counterweight smaller than the actual effect, the tendency of the combination being to flatten out the curves; for example, Series 12 gives results which are considerably higher than those of Table 10. It seems probable that there is some transfer from main driver to adjacent drivers through the equalizer bars when the counterweight is up and *vice versa* when it is down.

The effect of speed alone, as shown by the speed effect ratios, is quite uniform for the drivers and trailer and is closely proportional to the speed. The additional stress due to speed alone is quite moderate for all three wheels. The stress at the truck wheels increases rapidly, though the value at 60 miles per hour is not excessive. Fig. 17 gives the stresses at the wheels of the tender for Series 9, 10, 11, and 12 combined. The effect of speed is considerable, the average ratio of stress at 45 and 60 miles per hour to the stress at 5 miles per hour being 1.95 and 2.25, respectively.

The total effect ratio indicates that the effect of speed and counterbalance together does not differ much for the several drivers and the trailer, either for maximum values or generally for minimum values. Attention is called to the very high stress at the trailer at all speeds and to the considerable increase with speed—this too in results which average the data with the equalizer pin in the two positions. The stress in the third driver was also high. It should be borne in mind, of course, that the band of the plotted data gave frequent points 4 000 lb.

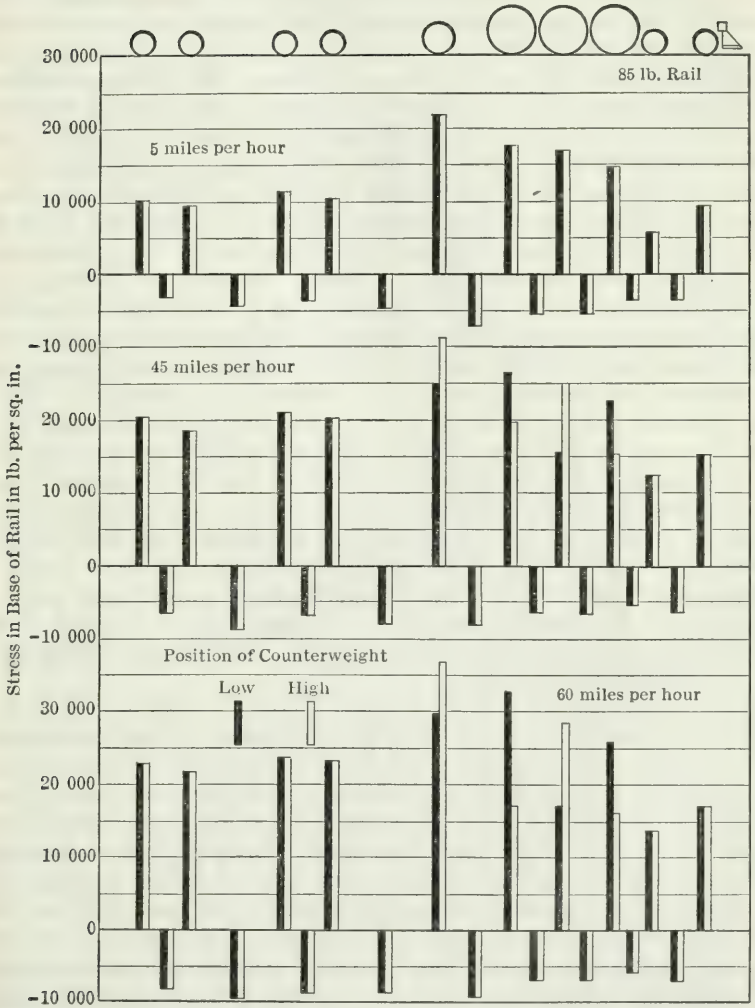


FIG. 17.—STRESS IN RAIL AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 9, 10, 11, AND 12 COMBINED, PACIFIC TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

per sq. in. higher than these values and that occasional values were found which are more than 10 000 lb. per sq. in. higher.

The Railroad Company lightened somewhat some of the rotating and reciprocating parts of Locomotive No. 1063 by cutting away metal. The weight given for the lightening of the rotating parts included 9 lb. for the rear end of the main rod and 54 lb. for the part of the side rod considered to be tributary to the main driver. The information furnished is that no change was made in rotating parts affecting the first and third drivers. The effect of decreasing the rotating weight at the crank pin by 63 lb. would be to increase the downward pressure at the main driver when the counterweight is down and to decrease it when the counterweight is up—to decrease the dynamic augment effect if the main driver is underbalanced and to increase it if overbalanced. The effect upon the two adjacent drivers would be to decrease the equivalent single load pressure somewhat for counterweight down, but the effect would be small. The data of Series 7 and 8 for stresses at the main driver are indefinite on the matter of effect of counterbalance; the points are quite scattered and it was found to be impracticable to draw a curve for a speed of 60 miles per hour, and that for 45 miles per hour was less regular than those found in other tests. The diagrams give the impression that the counterbalance effect at this wheel was small and rather variable for the different positions of the counterweight. For a speed of 60 miles per hour the calculated change in stress in rail at the main driver due to the 63-lb. change in counterbalance would be a decrease of 1 500 lb. per sq. in. when the counterweight is up and an increase of the same amount when it is down, and for a speed of 45 miles per hour a change of 900 lb. per sq. in., and it seems probable that such a change was effected.

15.—Test with the Mikado Type Locomotive.—The tests with the Mikado locomotive No. 1769 of the Illinois Central Railroad were made principally to find the effect of speed and counterbalance upon stress in rail for this type of freight locomotive. The results are concordant and generally definite and in these respects are more satisfactory than most of the tests which have been made. Sample plots of observed values of stress in rail are given in Figs. 18 to 22, which give also the curve of average stress throughout the revolution of the driver, the belt of most frequent values, and a line representing the mean of the stresses found in one revolution. Fig. 23 gives curves of average stress throughout a revolution for the several wheels at the four speeds. Fig. 24 shows the stress at high point and low point of counterweight taken from these curves. These values of stress in rail are given in Table 11, together with the mean value for the curve of average stress.

Table 12 gives the total ratio (combined speed and counterbalance effect), speed effect ratio, $\frac{S_m}{S_5}$, and counterbalance effect ratio, the

TABLE 11.—Stresses in Rail with Mikado Type Locomotive of the Illinois Central Railroad. Maximum, Minimum, and Mean Values from Curves.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.				Front truck wheel.
			4	Main.	2	1	
5	Mean value	15 800	16 500	14 200	14 200	15 600	8 800
25	Up	21 400	16 600	20 800	15 600	15 600	
	Down	15 600	24 000	12 000	20 300	22 400	
	Mean value	18 500	20 600	16 100	18 000	19 700	12 700
35	Up	22 400	15 600	25 000	13 000	13 000	14 600
	Down	18 200	25 000	10 400	23 400	25 000	12 000
	Mean value	19 000	20 100	17 700	18 200	19 000	13 100
45	Up	26 500	15 600	30 500	16 600	15 600	19 600
	Down	21 300	32 800	11 500	26 500	29 100	11 500
	Mean value	24 200	24 000	20 800	21 200	22 000	15 400
Calculated stress under static load		16 300	18 200	15 300	12 500	16 500	9 600
Calculated additional stress due to counterbalance at 45 miles per hour		200	4 300	3 200	3 200	4 300	400

nomenclature being the same as was used in the discussion of the tests with the Santa Fe and Pacific locomotives and the stress at 5 miles per hour being the divisor used in obtaining the ratios.

It will be noted that the stress at the first, second, and fourth drivers is greatest when the counterweight is down, and that the stress at the main driver is greatest when the counterweight is up. The stress at the trailer is greatest when the counterweights of the drivers are somewhat past their high position. The truck wheel shows the effect of counterbalance, the high stress in rail occurring when the counterweights of the drivers are slightly past the high position (about one-seventh of a revolution), and the corresponding low stress at the opposite position. The results indicate that the main driver is underbalanced for rotating parts, and that the other drivers are overbalanced.

The figures for counterbalance weights given by the Motive Power Department of the Illinois Central Railroad (see Table 3, page 1425), indicate the same condition of counterbalance for each driver, 263 lb. overbalance for rotating parts (to balance 60% of the reciprocating parts, considered to be taken by the four drivers equally). Not only is the position of counterweight of the main driver which gives the highest stress in rail the opposite of that obtained by analysis for the weights given in the table, but the counterbalance effect found for the several drivers is much larger than the calculated values. For the first, second,

TABLE 12.—Ratios of Maximum Stress, Mean Stress, and Additional Stress to Stress at 5 Miles per Hour, Mikado Type Locomotive of the Illinois Central Railroad.

Position of counterweight.	Trailer.		Driver Number.								Truck wheel.	
			4		Main.		2		1			
	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.	Up.	Down.
Average stress at 5 miles per hour assumed as basis of comparison	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
25 Miles per Hour.												
Total effect $\frac{S_{25}}{S_5}$	1.36	0.99	1.01	1.46	1.46	0.84	1.10	1.42	0.97	1.43	1.49	1.49
Effect of speed $\frac{Sm}{S_5}$	1.17	1.17	1.25	1.25	1.13	1.13	1.26	1.26	1.26	1.26	1.49	1.49
Effect of counterbalance (increase or decrease) $\frac{S_{25} - Sm}{S_5}$	+ 0.18	− 0.18	− 0.24	+ 0.20	+ 0.33	− 0.29	− 0.17	+ 0.16	− 0.29	+ 0.17	0.00	0.00
Calculated effect of counterbalance	0.00	0.00	− 0.07	+ 0.07	− 0.06	+ 0.06	− 0.08	+ 0.08	− 0.08	+ 0.08	+ 0.02	− 0.02
35 Miles per Hour.												
Total effect $\frac{S_{35}}{S_5}$	1.41	1.15	0.95	1.49	1.75	0.73	0.91	1.64	0.83	1.60	1.71	1.40
Effect of speed $\frac{Sm}{S_5}$	1.20	1.20	1.21	1.21	1.25	1.25	1.28	1.28	1.22	1.22	1.54	1.54
Effect of counterbalance (increase or decrease) $\frac{S_{35} - Sm}{S_5}$	+ 0.21	− 0.05	− 0.26	+ 0.28	+ 0.50	− 0.52	− 0.36	+ 0.36	− 0.39	+ 0.38	+ 0.17	− 0.13
Calculated effect of counterbalance	+ 0.01	− 0.01	− 0.14	+ 0.14	− 0.13	+ 0.13	− 0.16	+ 0.16	− 0.16	+ 0.16	+ 0.03	− 0.03
45 Miles per Hour.												
Total effect $\frac{S_{45}}{S_5}$	1.68	1.35	0.95	1.99	2.14	0.91	1.15	1.86	1.00	1.87	2.29	1.34
Effect of speed $\frac{Sm}{S_5}$	1.53	1.53	1.46	1.46	1.47	1.47	1.49	1.49	1.41	1.41	1.81	1.81
Effect of counterbalance (increase or decrease) $\frac{S_{45} - Sm}{S_5}$	+ 0.15	− 1.18	− 0.51	+ 0.53	+ 0.67	− 0.55	− 0.34	+ 0.37	− 0.41	+ 0.45	+ 0.48	− 0.47
Calculated effect of counterbalance	+ 0.01	− 0.01	− 0.24	+ 0.24	− 0.21	+ 0.21	− 0.26	+ 0.26	− 0.26	+ 0.26	+ 0.05	− 0.05

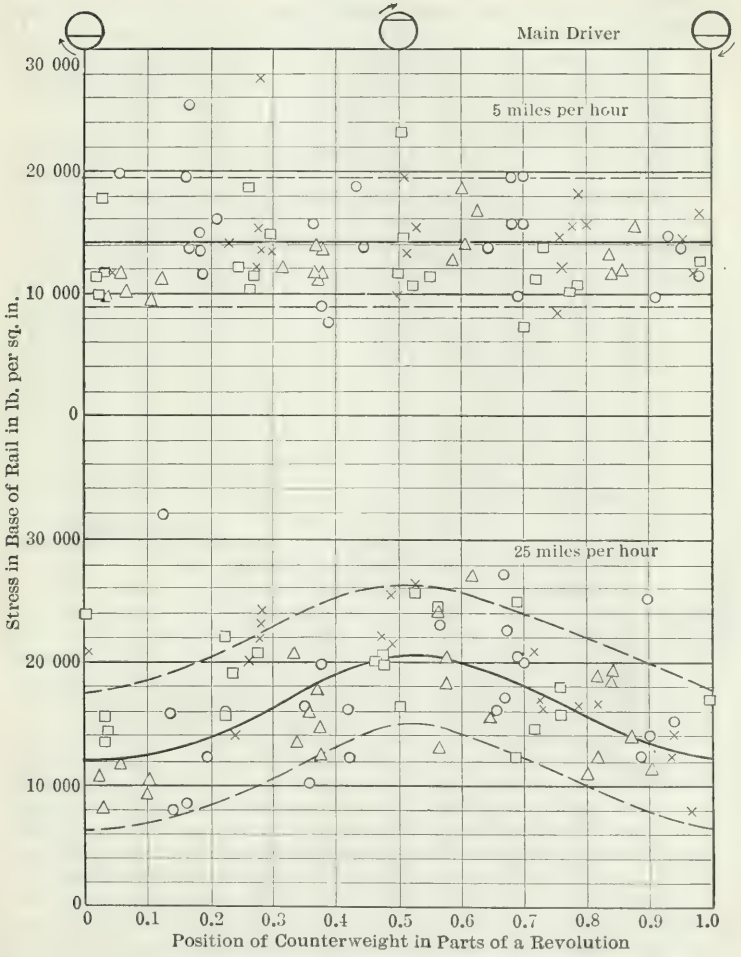


FIG. 18.—OBSERVED VALUES OF STRESS IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

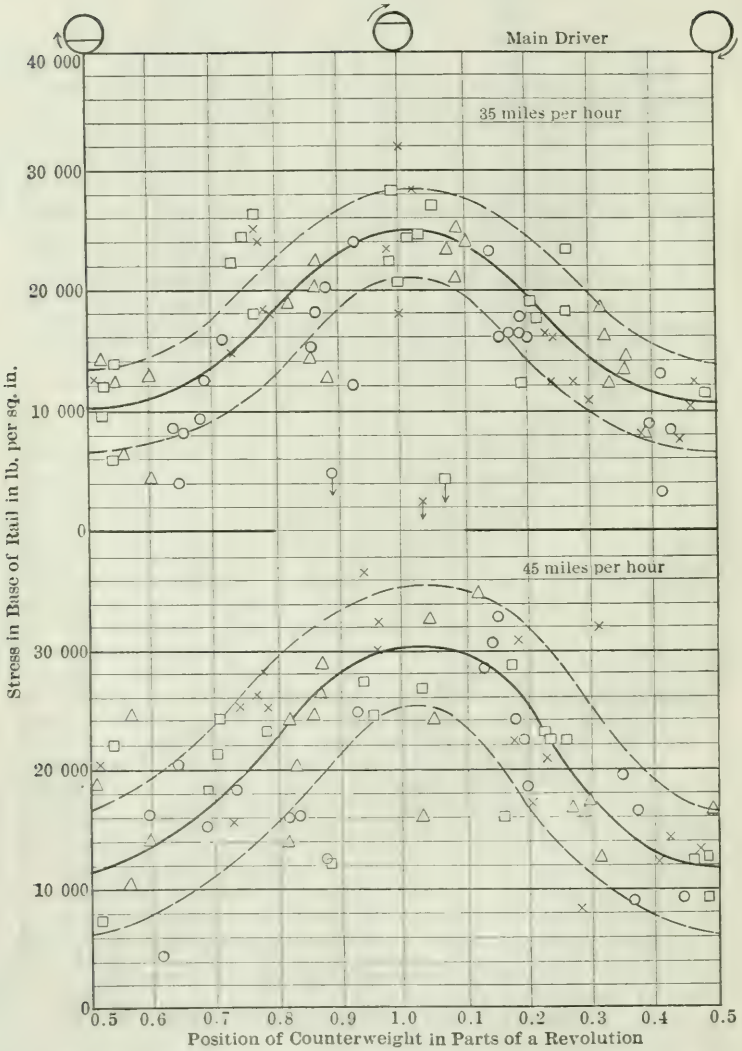


FIG. 19.—OBSERVED VALUES OF STRESS IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

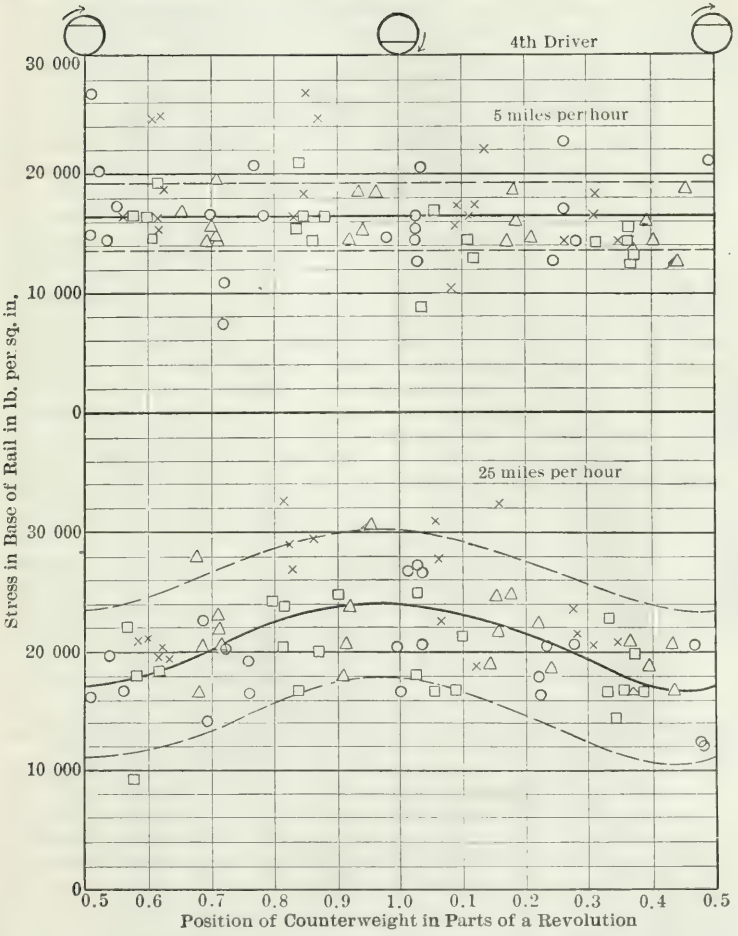


FIG. 20.—OBSERVED VALUES OF STRESS IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

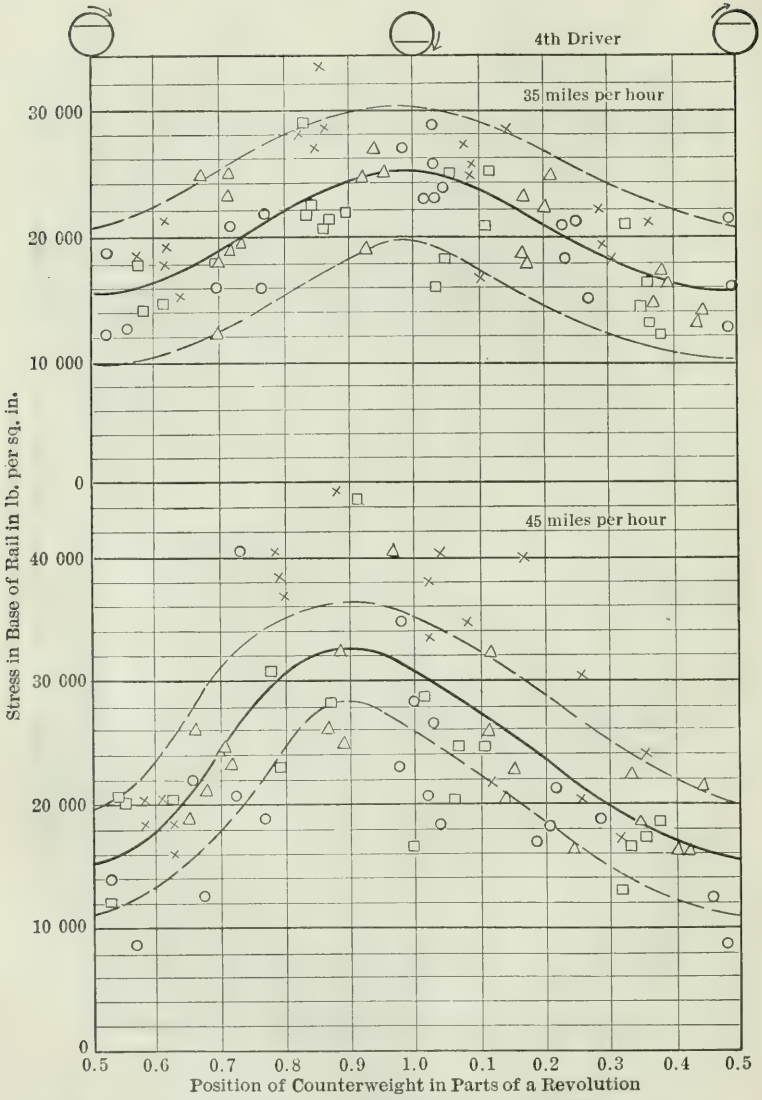


FIG. 21.—OBSERVED VALUES OF STRESS IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

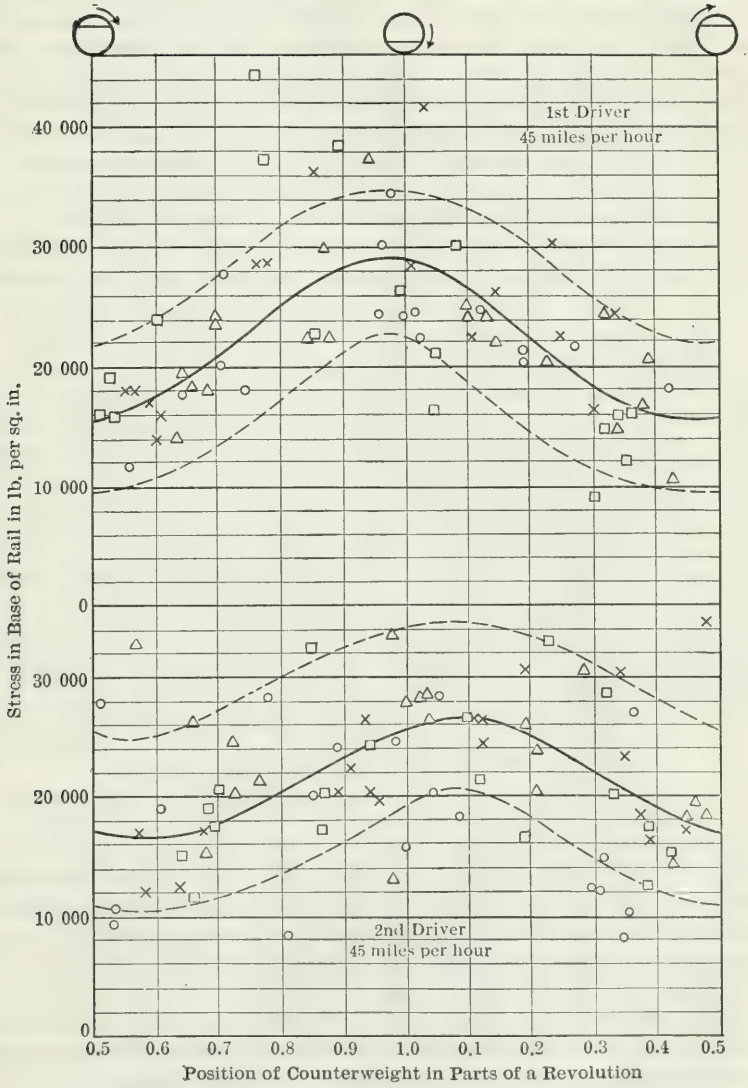


FIG. 22.—OBSERVED VALUES OF STRESS IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

and fourth drivers and for the speeds of 25, 35, and 45 miles per hour the average of the ratios expressing effect of counterbalance given in Table 12 is more than twice the calculated value. For the main driver at these three speeds the ratios expressing the counterbalance effect (0.33 at 25 miles per hour, 0.50 at 35 miles per hour, and 0.67 at 45 miles per hour) are three to four times the calculated values, and the high stresses occur when the counterweight is up instead of when it is down as would be expected from the counterbalance data given. A consideration of the planes in which the several rotating parts act only partly explains the discrepancy in the case of the main driver.

No explanation is offered for the remaining discrepancies. Possibly the weights of parts may not be correctly reported, and their distribu-

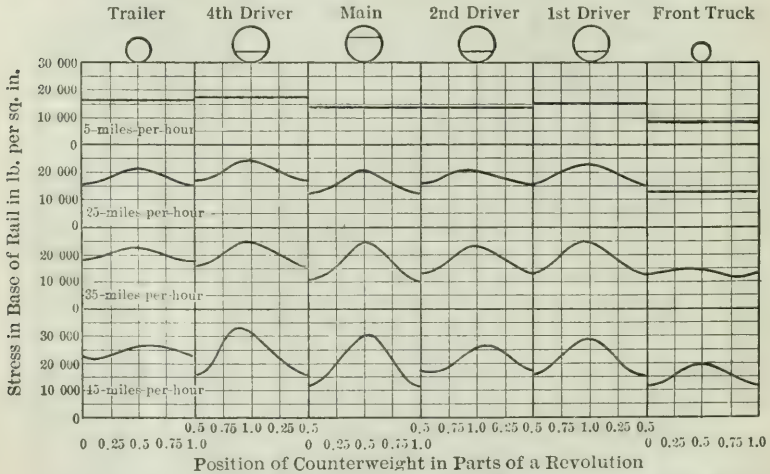


FIG. 23.—CURVES OF AVERAGE STRESS IN RAIL THROUGHOUT REVOLUTION OF DRIVER, MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

tion may also not be correct. Although the counterbalance effects are generally not large, the additional stress at the main driver at 35 miles per hour (the speed limit given by the railway regulations) is 50% as great as the stress at 5 miles per hour, and the stresses due to counterbalancing would be materially less if the counterbalance effects were in the same direction for each driver.

The effect of speed alone, as shown by the speed ratios in Table 12, is about the same for the several drivers. The ratios for truck wheel and for trailer are considerably higher at the higher speeds.

16.—Relation of the Stresses at the Two Sides of the Base of Rail.— In the first progress report it was shown that lateral bending of the rail occurred to a considerable extent; that most frequently the rail was bent outwardly at a point under a wheel and inwardly at a point between

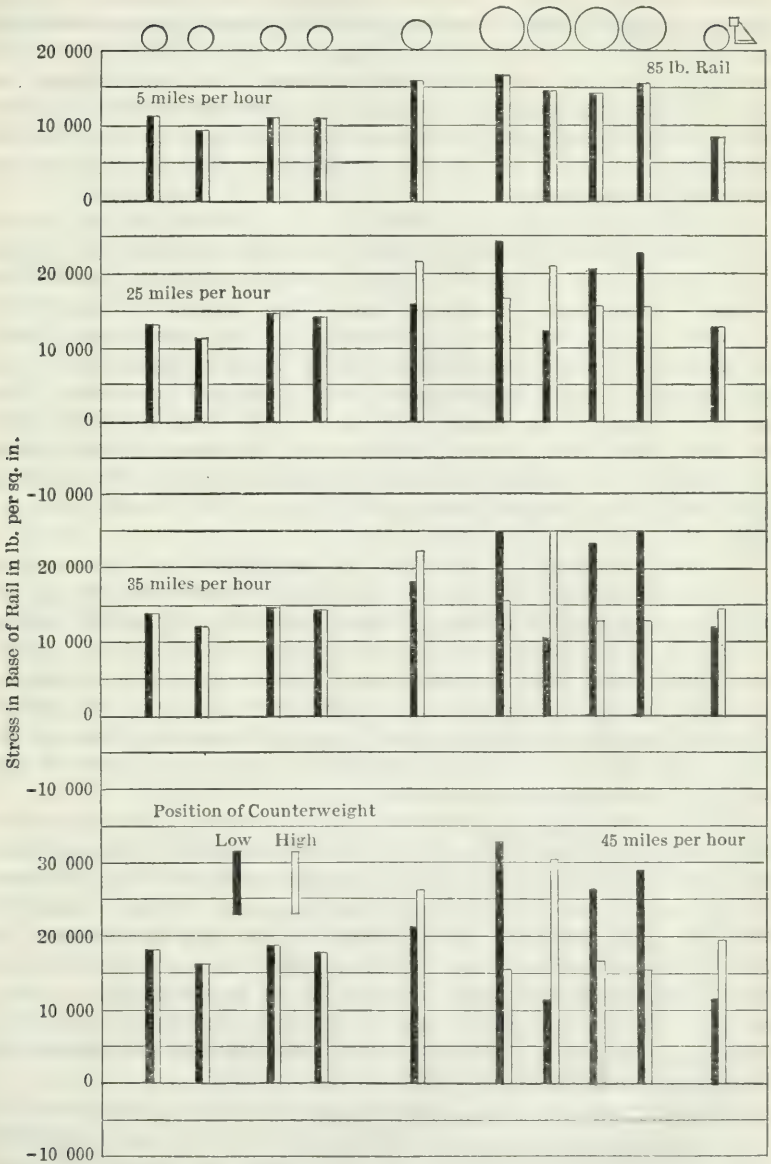


FIG. 24.—STRESS IN RAIL AT HIGH AND LOW POSITION OF COUNTERWEIGHT, MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

wheels, though a flexure opposite to this also was frequent; that the change in gauge of track accompanying this bending was measurable; that the bending occurs with the locomotive at rest and at low speeds as well as at higher speeds; and that the stress at the outer edge of the base of rail may occasionally reach three or more times that at the inner edge. An examination of the data of some of the tests described in this report has been made with a view of supplementing the information given in the first report.

A study has been made of Series 1 and 2 (with the Santa Fe type locomotive), Series 10 and 12 (with the Pacific type), and the series made on the Illinois Central Railroad (with the Mikado type). The stresses for the two sides of the base of rail were plotted separately according to the position of the counterweight, and curves were drawn to represent the trend and position of the points throughout the revolution for stress at both outer edge of base of rail and inner edge. The curves for stress at outer edge and at inner edge are quite similar, that for outer edge being higher than the curve of average stress in base of rail heretofore reported, and that for inner edge lower. The point of maximum stress for outer edge generally comes a little later in the revolution than that for inner edge.

It was found that the average of all the observations for the outer edge was greater than that for the inner edge. This conclusion applies at all speeds and at all positions of the counterweight for all the drivers and to the higher stresses found as well as to the lower. It also applies to the trailer and to the tender wheels. The average of the stresses at the outer edge for the several wheels ranges from, say, 20 to 50% greater than the average at the inner edge; that is, the average stress at the outer edge is from 9 to 20% greater than the mean stress in base of rail, which is the stress heretofore used.

This indicates that a general condition exists tending to give outward lateral bending in rail. No further investigation has been made to determine what that condition is. It may be noted that the difference in stress at the two sides of the rail was as great, or greater, under the main driver as under the other drivers. In the case of the Santa Fe type this driver has no flange and was slightly worn. While the number of observations giving the higher stress at the outer edge was considerably greater than that at the inner edge, the number having the opposite relation was by no means small, showing that the swaying of the locomotive laterally is a factor in the problem.

A similar relation exists at points between wheels in the region of negative bending moment, though here the lateral bending is generally in the opposite direction from that at the wheel and the average stress at the inner edge is greater than that at the outer edge.

More important than the average value at the edges of the base of rail are the relative values of the individual observations. The ratios of

the stress at edge to the mean stress in base of rail were plotted. A study of these shows that in 15 to 20% of the observations, the stress at the outer edge is 33% or more greater than the mean stress in base of rail, in a few cases reaching an excess of 50% or more, and that this holds good throughout the revolution of the driver. An excess of 20% is quite common. A stress at inner edge 33% greater than the mean stress was found in, say, 5 to 8% of the observations. As an example of a large difference in stresses at the two edges, though not of a high ratio of stresses, values for the main driver of the Santa Fe type at a speed of 50 miles per hour may be cited, 36 400 lb. per sq. in. at the inner edge, and 55 000 lb. per sq. in. at the outer.

The lateral bending in rail has some bearing on the design of rail section and on the design of tie-plates. It will be seen that for the rail sections used, a ratio of stress at outer edge to mean stress in base of rail having a value of 1.33 means that the lateral bending moment in the rail is about 8% of the vertical bending moment required to develop the mean stress. It should be noted that the results herein presented were obtained with track which may be said to be in ordinarily good condition; the results given in the first progress report for track in poorer condition and for track with decayed ties show that the amount of lateral bending may be materially greater and that therefore the condition of the track is an important element in the problem.

17.—General Discussion.—In Figs. 25, 26, 27, and 28 the ratios for total effect, speed effect and counterbalance effect given in Tables 6, 8, 10, and 12 have been plotted. These ratios show the relation between the stress in rail at the several higher speeds and the stress in rail existing at a speed of 5 miles per hour. In all the series the total effect ratios are concordant and show a marked increase of stress with speed, the increase being faster than the first power of the speeds. The method used in making a division of this total effect ratio into effect of speed alone and effect of counterbalance is of course not accurate, and it may be expected that a large counterbalance effect will result in an increased speed effect as well. Fig. 29 gives the ratios for the average speed effect for the tender wheels of the three types of locomotives used, as well as the values of the average stresses at these wheels.

It will be noted that the increase in stress in rail at drivers and trailer beyond 5 miles per hour due to speed alone, varies almost directly as the increase in speed, though in a few instances it is at a somewhat faster rate. Averaging the values of the ratios at the drivers and trailer and for comparison using a speed of 50 miles per hour in all cases, it is seen that for this speed the increase in stress in rail due to speed alone over that at 5 miles per hour will be about 70% for Series 1 and 50% for Series 2 (average of 60% for the Santa Fe type locomotive used), about 35% for the Pacific type locomotives, and about 50% for the

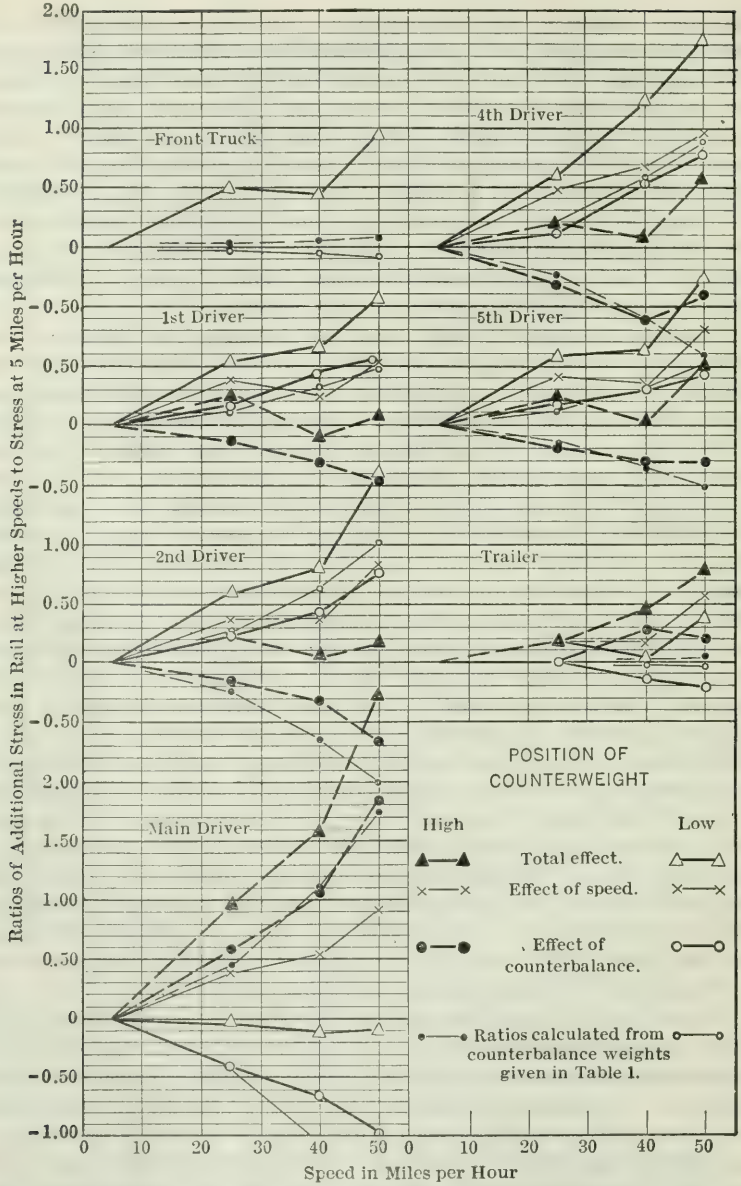


FIG. 25.—RATIOS OF TOTAL EFFECT, SPEED EFFECT, AND COUNTERBALANCE EFFECT TO STRESS AT 5 MILES PER HOUR, SERIES 1, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

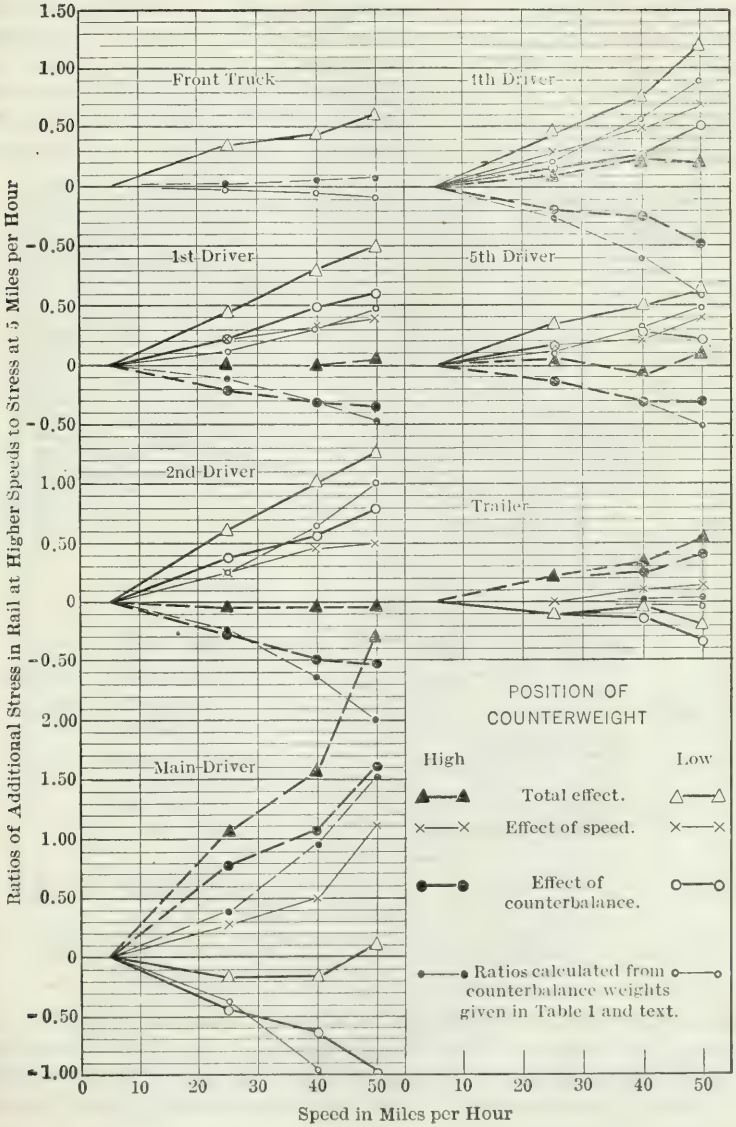


FIG. 26.—RATIOS OF TOTAL EFFECT, SPEED EFFECT, AND COUNTERBALANCE TO STRESS AT 5 MILES PER HOUR, SERIES 2, SANTA FÉ TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

Mikado type locomotive. At the tender wheels, the stresses in rail increase almost directly as the increase in speed. The increase in the stresses at 50 miles per hour over those at 5 miles per hour at the tender wheels is approximately as follows: Series 1, 50%, Series 2, 100% (an average of 75% for the Santa Fe type locomotive); Pacific locomotive 100%; and Mikado locomotive, 75 per cent. It is seen that the ratios found for the drivers are in general less than those for the tender

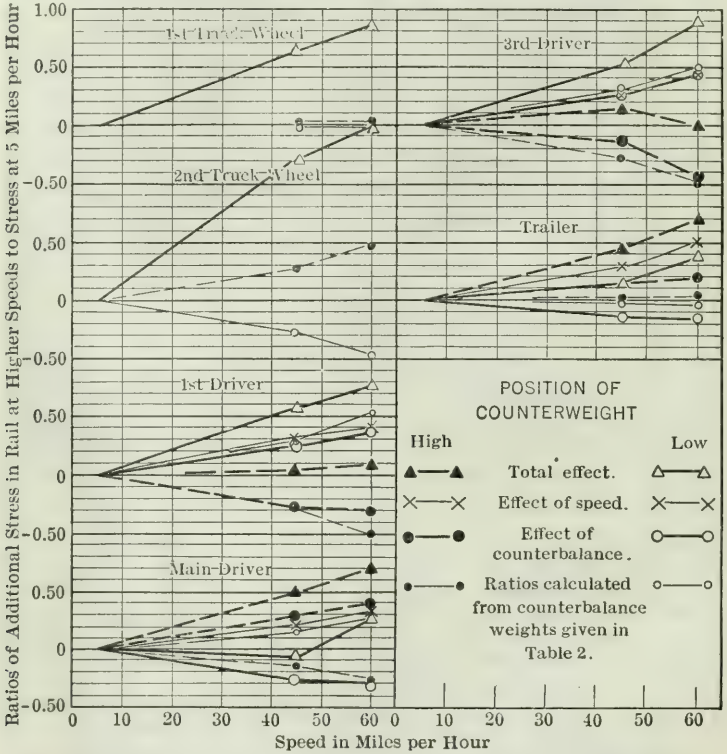


FIG. 27.—RATIOS OF TOTAL EFFECT, SPEED EFFECT, AND COUNTERBALANCE EFFECT TO STRESS AT 5 MILES PER HOUR, SERIES 9, 10, 11, AND 12 COMBINED, PACIFIC TYPE LOCOMOTIVE OF THE ST. LOUIS-SAN FRANCISCO RAILWAY.

wheels (the ratios do not include the effect of counterbalance), and the increase at the tender wheels varies considerably for the different locomotives. In some of the locomotives (particularly the Santa Fe type) the increased stress at some of the wheels is materially higher than the average values quoted above.

The counterbalance effect at low and high position of counterweight, with both underbalance and overbalance, is brought out in Figs. 25, 26, 27, and 28. It appears that with the main driver underbalanced

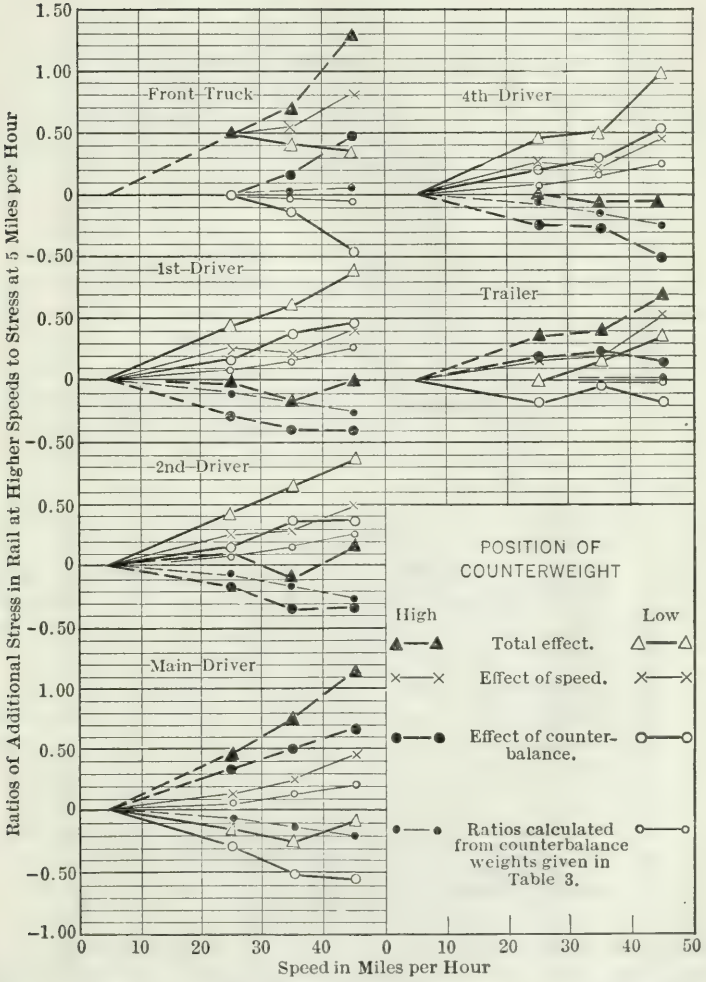


FIG. 28.—RATIOS OF TOTAL EFFECT, SPEED EFFECT, AND COUNTERBALANCE EFFECT TO STRESS AT 5 MILES PER HOUR, MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

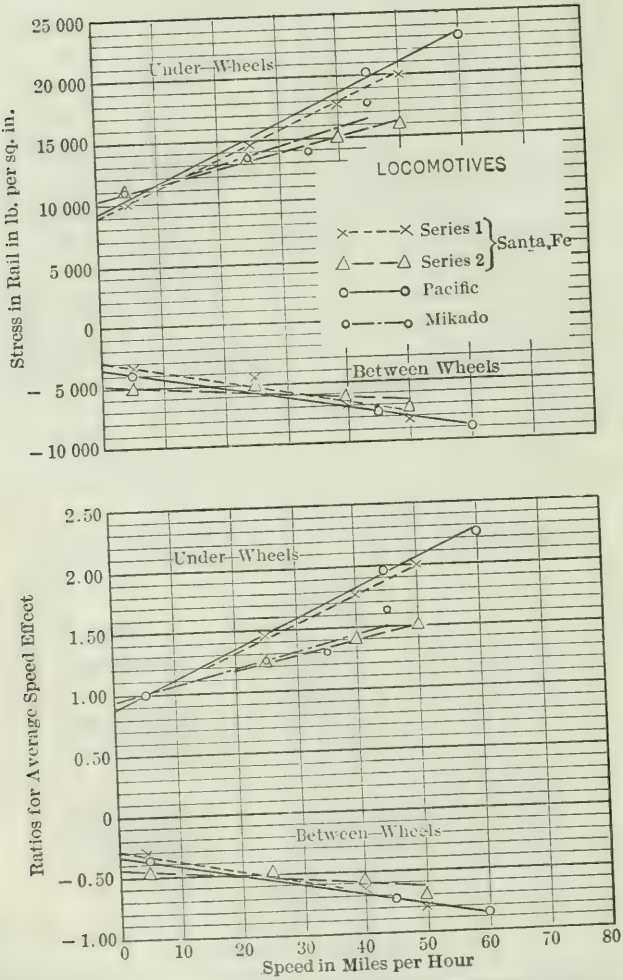


FIG. 29.—AVERAGE STRESSES AND RATIOS FOR AVERAGE SPEED EFFECT FOR THE TENDER WHEELS OF THE THREE TYPES OF LOCOMOTIVES.

and the other drivers overbalanced there is some transfer of pressure through the equalizer bars, tending to reduce somewhat the stresses due to counterbalance, but the effect is not very definite and seems to be uncertain. The stress due to counterbalance increases much faster than the increase in speed, though not always as fast as the square of the speed. The average increase in stress at the several drivers due to counterbalance effect, as indicated by the ratios in the diagrams, for a common speed of 50 miles per hour over the stress at 5 miles per hour is about 90% for Series 1 and 80% for Series 2 (average 85% for the Santa Fe type locomotives used); 30% for the Pacific type; and 60% for the Mikado type. The increase in stress at the main driver in each case is greater than this average, and with the Santa Fe type very much greater.

Points representing the calculated counterbalance effect ratio are also given in Figs. 25, 26, 27, and 28; they are based upon the counterbalance weights given in Tables 1, 2, and 3 without consideration of the effect of the planes of rotation of the rotating parts being in different planes or of other disturbing elements. It will be noted that in a number of cases there is a considerable discrepancy between the observed and the calculated ratios.

The following general observations are made:

It is evident that there is a decided advantage to the track in making the counterbalance of the several drivers differ but little from each other and from that condition which will give the best results, even though opposing conditions of counterbalance in the main driver and the adjoining drivers may have some neutralizing effect. An underbalance on one driver and an overbalance on an adjoining one will give a very undesirable cumulative effect.

It is of course to be expected that when the locomotive is working the downward pressure of the main driver will be increased over that obtaining when the steam is shut off as was the case in the tests here discussed.

For the Santa Fe type locomotives tested the stresses at the main driver were much greater than at the other drivers.

Considering the stresses developed in the tests it is not strange that great injury to track is done by poorly balanced locomotives run at high speed. Even at lower speeds, where kinking of rail does not result, the punishment of the track is much greater than will be caused by well-balanced locomotives. The effect of counterbalance and speed is to add greatly to the load to be carried by rail, tie, and ballast.

Effort may well be made to reduce the counterbalance effect on stress in track by use of the best methods of design of locomotive and by the use of material which will permit a minimum of counterbalance effect in both rotating and reciprocating parts.

It is important that care be taken to determine the actual weights and position of counterweights and the actual properties of the side rods and connecting rods. Care should be taken to learn whether the assumed division of load among the drivers in the completed locomotives is obtained.

Locomotives will give very different stresses in rail at the different wheels according to the wheel spacing, the division of load, and the effect of adjoining wheels.

It is apparent from analysis that for the spacing of wheels in the three types of locomotives the stress at the trailer under static load or at low speeds will be relatively large if the weight on the trailer is greater than, say, 75% of the average weight on drivers; and the tests would indicate that for locomotives with even fairly good counterbalancing a discrepancy at low speeds between the stress at the trailer and the stress at a driver, due to a relatively heavy load on the trailer, may be expected to exist also at higher speeds.

It was found that the wheels of the tenders of the different types of locomotives gave quite different effects of speed on the stress in rail.

Attention may well be called to the method herein used in calculating the stress in rail for a combination of driver loads. The method ordinarily adopted in taking into account the counterbalance effect uses the dynamic augment for the given driver only. In getting the stress-producing bending moment in the rail, however, it is necessary to take into account the dynamic augment also of the adjoining drivers. With the main driver underbalanced and the adjoining drivers overbalanced the pressures given by the adjacent drivers are especially influential and the effect of the combined loads should not be overlooked. It should be added, too, that the effect of counterbalancing upon bending moment in rail is different from the effect upon rail depression and pressure on rail—these must be calculated separately.

In the discussion of counterbalance, and the calculation of the stress produced in the rail, no attempt has been made to go into the complications of the problem involved in the yielding of the locomotive springs and of the track acting as a spring, the effect of the rotating parts not being in the same plane, and other disturbing elements.

The tests were made on 85-lb. rail. It is felt that the effect of speed and counterbalance in producing stress in rails of other weight will follow in general the results found with the 85-lb. section. The increase in stress at a given speed over the stress at 5 miles per hour may be considered to be produced by additional loads at the drivers, due to speed and counterbalance, and the amount of these additional loads may be calculated by the use of the analytical methods outlined in the report. The loads thus found for speed and counterbalance effect

may be used in judging of the effect of speed and counterbalance with rails of other sections. The calculations for stress in a section may be made on the basis of the analysis for bending moments given in the first progress report, the section modulus of the given rail being used in calculating the stress developed by the bending moment found.

It is evident that wheel spacing, distribution of load among drivers and trailer, and counterbalancing have a marked influence upon the stresses developed in rail and also in the track structure below the rail. Adequate consideration therefore should be given to these elements in designing locomotives if the track is not to be called upon unduly to carry loads and to resist stresses which are unnecessary or excessive. The uncertainty which may exist concerning exact conditions of counterbalance is a matter which warrants greater attention.

III. Track Depressions.

18.—Depression of Track under Load and Modulus of Elasticity of Rail Support.—In the first progress report of the Committee the general question of the depression of track under load was considered and the effect of the make-up and condition of the track structure upon stresses in rail and distribution of load among the ties was discussed. In connection with the tests to determine the flexural action of ties in track (presented under "IV.—Depression, Flexure, and Bearing Pressure of Cross-Ties"), made since that report was prepared, tests have been made to obtain the depression of the track under the conditions which obtained when these tests of ties were made. A purpose of making these profiles of rail depression was to determine the proportion of the load applied to the rails which may be considered to be taken by a single tie. The tests served also to show the relative effects of different amounts of load and something of the general condition of the track.

Under the best conditions, with freshly tamped track, the depression at any tie may be expected to be nearly proportional to the load applied to the rail and likewise proportional to the load taken by the individual tie. For track not recently tamped or in poor condition there will be a slight play between the tie and the ballast along parts of its length, and part of the resistance before the tie takes bearing along its length will be due to the bending of the tie under the action of the lighter loads. These and other influences found in track not in the best condition make the line representing the relation of track depression to applied load vary from a straight line through at least a part of its length.

In Fig. 30, the relation between tie depression and load on tie for track in best condition is illustrated at (*a*). The vertical distances represent the depression of rail; the horizontal distances are proportional to the load on the track. This condition gives a constant modulus

of elasticity of rail-support—the term defined in the first progress report as the pressure per unit of length of each rail required to depress the track one unit and which was found to be of considerable service in analytical investigations.

At (*b*) is illustrated the case where a light load produces considerable depression and greater loads result in additional amounts of depression which are proportional to the increments of load. The distance from the origin to the point where the line passing through the points intersects the vertical axis may be considered to represent in part the play or space between the bottom of the tie and its bed when

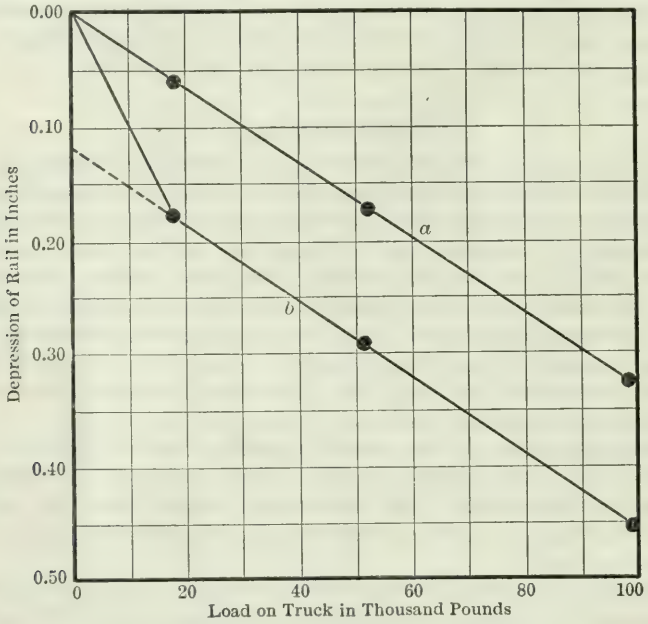


FIG. 30.—TYPICAL LOAD-DEPRESSION DIAGRAM.

the tie is not under load, though of course there is some bearing resistance by the tie for track depressions less than the amount of this intercept, and a part of the total play may be between the rail and the tie. In general the direction of the line through the points obtained by the several loads represents the load-depression relation and is closely parallel to the line found for well-tamped track. For general purposes the direction of this part of the line may be used in determining the value of the modulus of elasticity of rail-support.

19.—Track-Depression Tests on Illinois Central Railroad.—In the tests of track depression made in 1918 the load was furnished by cars loaded in such a way as to give the desired weight on the truck at one

end of the car. The end of the car to be used in the test was weighed on track scales. In the use of the data of the tests it is assumed that each wheel of the car truck carried one-fourth of the weight so found. In the tests on the Illinois Central Railroad three loads were used, 100 000, 53 000, and 18 000 lb. on a single truck; that is, 25 000, 13 250 and 4 500 lb. on each of the four wheels of the truck. The smallest load was taken as being sufficiently heavy to bring the tie to a full bearing on its bed and to eliminate any play which may exist between rail and tie and tie and ballast. The wheel base of the truck was 5 ft. 6 in. The distance between trucks (31 ft. center to center for the car giving loads of 100 000 and 53 000 lb. and 29 ft. for the one giving 18 000 lb.) was such that the load on the trucks at the farther end of the car did not have a large effect upon the track depression at the middle of the length of the car. The center of the truck was placed at the middle of the length of the rail, except on Section *K* and on cinder ballast (alternate joints) where the load was placed at the quarter points of the rail.

The level bar described in the first progress report was used in taking the levels along the rails for zero load and for the other loads from which the track depressions were computed. The method of making a test was the same as that described in the first progress report. A fixed reference stake was placed 80 in. from the rail and levels were carried from this bench mark to the rail by means of the level bar for each set of levels. A set of levels along the portion of the rail measured was first taken for zero load. One of the loads was then applied and levels were run over the stretch of rail, and this was followed with measurements at the remaining loads. In all cases levels were run on both rails.

The tests were made on the main line of the Illinois Central Railroad about two miles north of Champaign, Ill., on the track previously prepared for the track tests and on the sections of track marked *B*, *C*, and *K* in Fig. 26 of the previous report. The ties were the 6 by 8-in. by 8-ft., sawed, oak ties furnished for the test track in 1914. The rail on Sections *B* and *C* was the 85-lb. rail of Am. Soc. C. E. section, the same as was in the track in the previous tests, and that on Section *K* was the 125-lb. rail also used in the other tests. Besides these tests, a test was made on the north-bound freight lead shown in Fig. 26 of the previous report, which has 85-lb. rail and 12 in. or more of cinder ballast, and the usual run of hewn oak ties with a few sawed ties.

The main track had not been tamped for some time, a year or more, and although only passenger trains used it ordinarily, the track was not in good surface condition. The track depressions found and the flexural action of the ties go to show looseness and play between tie and ballast. Attention should be called, too, to the scantiness of the shoulder

of ballast on the outer side of the south-bound track (see Fig. 43), particularly on Section *B*, which gave little support to the ties at this end. Altogether, however, the track was not in bad riding condition and may be said to be representative of much well-made track at times when it has received the effect of traffic over a considerable period after tamping. At one location, *B*-2, the track was tamped slightly between tests but not enough to improve its condition materially.

On Section *B*, tests were made at two locations (adjoining rails) which will be designated *B*-1 and *B*-2. This section has 12 in. of stone ballast. On Section *C*, the two locations (also on adjoining rails) will be designated *C*-1 and *C*-2. This section has 6 in. of stone ballast. At Section *K* the ballast is 24 in. deep.

Figs. 31, 32, 33, 34, and 35 give the track-depression profiles. The loads given are the total loads on a single truck. The levels were taken for a length of 40 ft. or more, depending upon the location of the joints, and the results for the full length are given for Location *K*, *C*-1, *C*-2 and cinder ballast; for the other locations the profiles for a shorter length are shown, including all the ties which carry loads of any amount. In Figs. 32 and 33 load-depression diagrams have been plotted below the profiles for different points along the rail, the horizontal distances being proportional to the load on the truck; the intercept on the vertical axis of the straight line used is taken as the vertical play in the track. In several cases the measurements for the east rail were found to be inconsistent and unreliable, and these have been discarded.

In making the calculations for tie reactions, the amount of the play found from the load-depression diagrams was subtracted from the observed tie depression; the total load was divided by the sum of the net depressions for all the ties carrying load; the product of this quotient and the net depression of a tie was taken as the load or reaction of that tie. If the play found for different parts of the track-depression profile was fairly uniform the average play was used. In some cases the play differed considerably at points along the profile and it was necessary to use that found for each tie. Generally speaking, the length of rail considered included all the ties showing a positive net depression. As there was some negative reaction on ties beyond these limits the actual load on the ties considered was in reality greater than the applied load.

In Table 13 the tie reactions are given for the several tests. The values are given in terms of the load on truck at the end of the car used. The ratios given in the table are more convenient for making comparisons than weights. Values found from the higher loads are given; for the low load the effect of the tie play is so great that comparison based on this load would not be especially useful. If gross depressions had

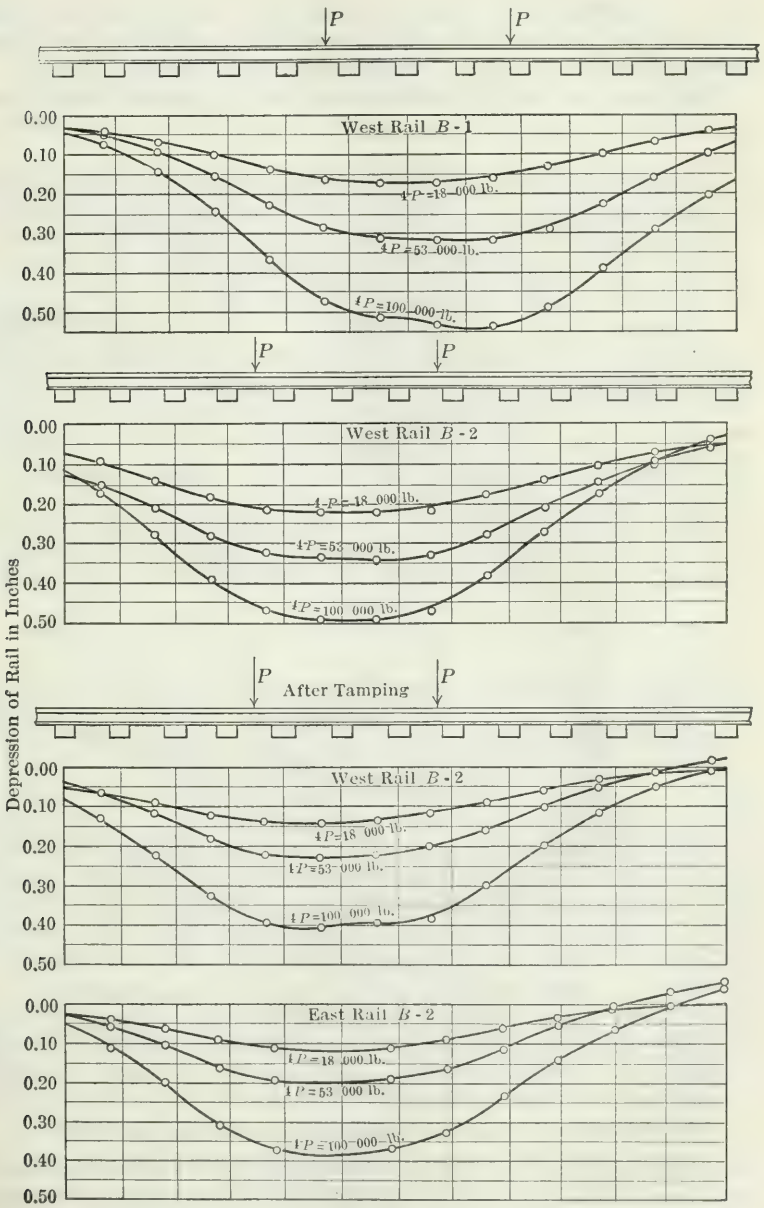


FIG. 31.—TRACK DEPRESSION PROFILES ON ILLINOIS CENTRAL RAILROAD.

been used instead of net depressions the tie reactions throughout the middle of the track-depression profile would be smaller than the values given in Table 13 and those away from the loads would be greater.

The values in Table 13 indicate that for this two-axle loading and 85-lb. rail the tie reaction at the wheel is from 13 to 16% of the load on

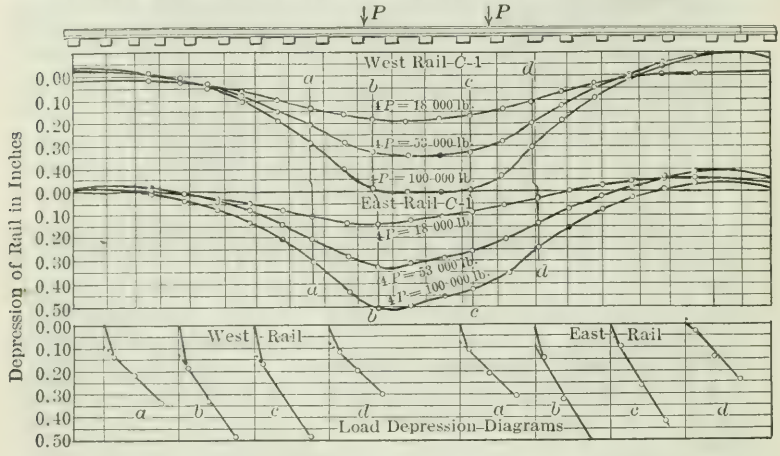


FIG. 32.—TRACK DEPRESSION PROFILES ON ILLINOIS CENTRAL RAILROAD.

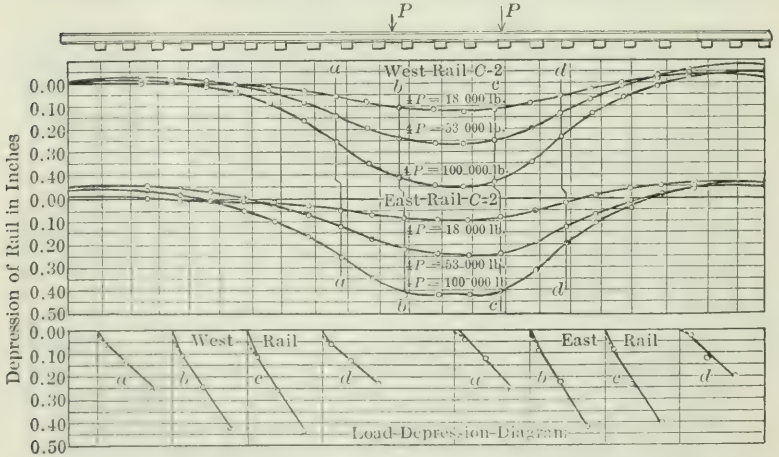


FIG. 33.—TRACK DEPRESSION PROFILES ON ILLINOIS CENTRAL RAILROAD.

the truck, and at a point midway between wheels the tie reaction may be 1½% more. The value derived by means of track-depression formulas found by the analysis given in the first progress report for the conditions of this test is about 15% at the wheel and 16% midway between

wheels. The depression away from the wheels also agrees fairly well with that determined by the analysis. Still farther away, in the region where negative pressures have more influence in the analysis, it cannot be expected that the conditions of the track will be those assumed in the analysis.

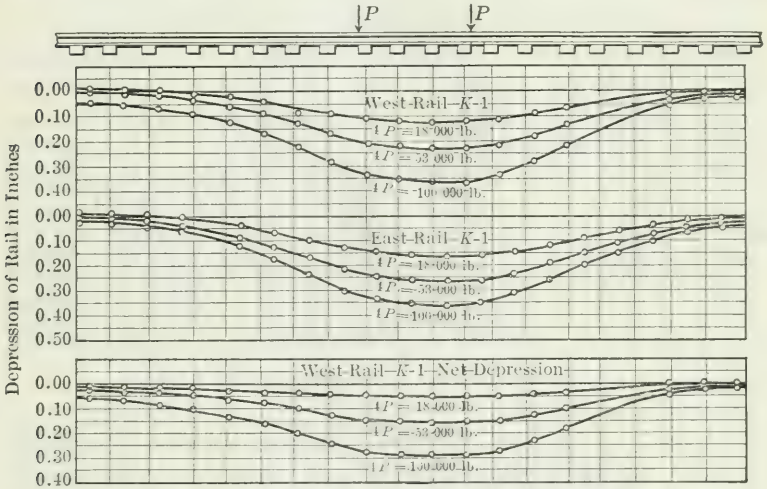


FIG. 34.—TRACK DEPRESSION PROFILES ON ILLINOIS CENTRAL RAILROAD.

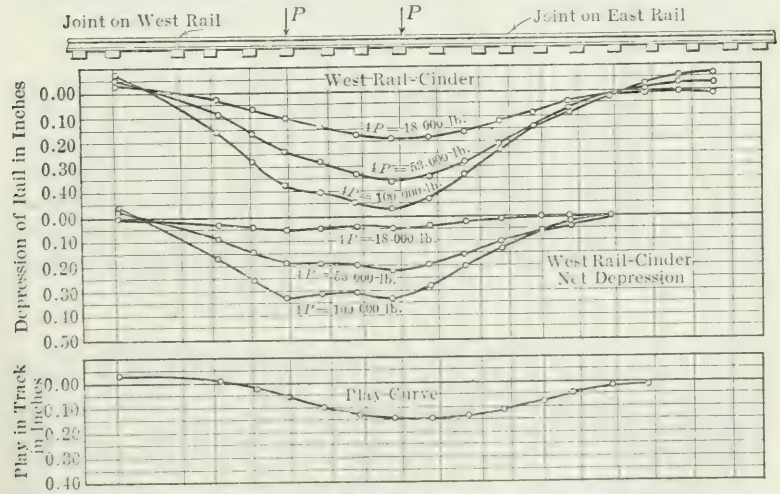


FIG. 35.—TRACK DEPRESSION PROFILES ON ILLINOIS CENTRAL RAILROAD.

The values at and between loads given in Table 13 are somewhat greater for a truck load of 53 000 lb. than for 100 000 lb., although the difference in ratios is not large. For the axle load of 18 000 lb. the ratio for the maximum tie reactions is still somewhat greater. If the partial

resistance corresponding to the first part of the depression of the track is taken into consideration, the difference in the ratios corresponding to the three loads will become still less. For most purposes it is sufficient to consider that the proportion of load taken by a tie is independent of the total load applied.

TABLE 13. – TIE REACTIONS IN TERMS OF LOAD ON TRUCK.
ILLINOIS CENTRAL RAILROAD.

Location of Tests	Wt. of Rail Lb. per Yd.	Size of Tie in Inches	Depth of Ballast	Total Load = 53 000 lb.									
B-1	85	6 x 8	12 in.		0.02	0.07	0.11	0.14	0.16	0.16	0.14	0.12	0.08
B-2	85	6 x 8	12 in.		0.01	0.08	0.15	0.17	0.18	0.18	0.15	0.08	0.00
B-2	85	6 x 8	12 in.		0.02	0.09	0.16	0 19	0.18	0.17	0.13	0.06	0.00
C-1	85	6 x 8	6 in.		0.05	0.11	0.17	0.18	0.17	0.15	0.11	0.05	0.01
C-2	85	6 x 8	6 in.		0.04	0.08	0.12	0.15	0.16	0.16	0.14	0.10	0.05
				Total Load = 53 000 lb.									
K	125	6 x 8	24 in.	0.00	0.00	0.00	0.13	0.14	0.15	0.15	0.14	0.10	0.06
Cinder	85	6 x 8		0.00	0.00	0.04	0.10	0.15	0.19	0.21	0.17	0.09	0.05
				Total Load = 100 000 lb.									
B-1	85	6 x 8	12 in.		0.03	0.07	0.11	0.14	0.15	0.15	0.14	0.12	0.09
B-2	85	6 x 8	12 in.		0.03	0.08	0.14	0.17	0.17	0.17	0.14	0.08	0.02
B-2	85	6 x 8	12 in.		0.05	0.10	0.14	0.16	0.16	0.15	0.13	0.08	0.03
C-1	85	6 x 8	12 in.		0.06	0.10	0.16	0.17	0.16	0.15	0.12	0.05	0.02
C-2	85	6 x 8	12 in.		0.06	0.10	0.13	0.14	0.15	0.15	0.13	0.09	0.05
				Total Load = 100 000 lb.									
K	125	6 x 8	24 in.	0.03	0.06	0.10	0.12	0.14	0.14	0.14	0.12	0.09	0.06
Cinder	85	6 x 8		0.00	0.00	0 09	0.14	0.16	0.17	0.18	0.15	0.07	0.03

As would be expected, the stiffer rail at Location *K* gives a wider distribution of load and a smaller maximum tie reaction. By the analysis for the conditions of the test, the reaction of the tie at a wheel is calculated to be 13% of the load on the truck. The depression profiles for this location are regular, and nearly the same for the two rails, as

might be expected with the stiffer track. On the other hand, the depression profiles for the freight track (cinder ballast) are irregular and uneven, the load-depression curves deviate considerably from a straight line, and the conditions are such as to give uneven loads on ties and to increase the stress in rail over that which would be found in good track.

The amount of the play between rail and tie and tie and ballast shown by the load-depression diagrams ranged from 0.06 to 0.18 in. (untamped track). On some of the sections, the amount varied considerably throughout the length carrying the load, say, 0 to 0.10 in., showing a great diversity in the bearing of the several ties. The condition of the bearing at the two sides of the track also differed considerably. These facts will be found to be significant when the discussion of the flexure of ties is made. It is true, of course, that in places away from the test sections, which had ties of uniform dimension and spacing, the track play was greater and more variable, as could be observed at the time trains were passing.

The values of u, the modulus of elasticity of rail-support, obtained from the test data for Locations B and C, range from 900 to 1 300 lb. per in. per in., with an average value of 1 100. The value at Location B-2, after the track had been tamped slightly, was about 1 200. The value at Location K (24 in. of ballast not recently tamped) was about 1 200. The value for the test on cinder ballast was 1 000, although it must be borne in mind here again that this track was not in good surface.

20.—Track Depression Tests on Chicago, Milwaukee and St. Paul Railway.—The tests for track depression on the Chicago, Milwaukee and St. Paul Railway were made in the manner already described for the tests on the Illinois Central Railroad. The loads on the truck at one end of a car were 100 000, 68 000, and 37 000 lb., which were assumed to be equally divided among the four wheels of the truck. The wheel base of the truck was 5 ft. 6 in., and the distance center to center of trucks was 28 ft. 10 in., 29 ft., and 25 ft. 11 in., respectively, for the cars giving the loads named above. As the rails were laid with alternate joints, the center of the trucks was placed at the quarter points of the rail.

The tests were made on the main line of the Milwaukee Division and on the Janesville Division, a branch line. The tests on the main line were made at two points (see Fig. 36), Q on the east-bound main track about a mile south of the station at Deerfield, Ill., (east in railroad parlance) and O and P on the east-bound main track a quarter of a mile nearer the station. In each case passing tracks permitted traffic to be routed so that the tests were not interfered with. The tests on the Janesville Division were made on the single track a mile west of the station at Libertyville, Ill.

On the main-line track at Deerfield (called by the Railroad Company Class A track) the rail was 100-lb., Am. Soc. C. E. section. The ties were mostly hewn red oak ties, zinc-chloride treated, though perhaps 20% were sawed ties which had replaced old ties. The hewn ties were variable in width ranging from 8 to 11 in.; the sawed ties were 8 in. wide. Part were 7 in. thick and 8 ft. 6 in. long and part 6 in. thick and 8 ft. long. The ties were spaced about 20 in. center to center (twenty to a rail length of 33 ft.). The spacing was fairly uniform. The ballast was bank-run gravel, 45% of which passes a ¼-in. sieve. The depth of ballast was about 36 in. At the side next the other main track, and also on the other side of Location *Q*, the surface of the ballast ran level from top of tie of track tested to top of the tie of the adjacent track. At Location *O* and *P*, on the outer side the ballast extended at level of the tie for a distance of 15 in. from the end of the tie and then sloped down 10 in.

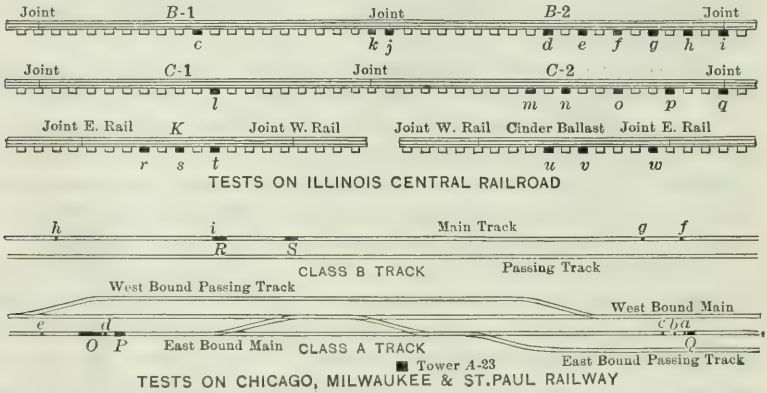


FIG. 36.—LOCATION OF TRACK DEPRESSION TESTS AND OF TIE FLEXURE TESTS.

in 2 ft. (see Fig. 44), so that in all cases there was adequate opportunity for full support at the end of the tie. The track surface was in good condition, but it had not been tamped for some months. The roadbed at Location *Q* was not much above the level of the ground; Location *O* and *P* was on an embankment of considerable age.

On the branch-line track at Libertyville (called by the Railroad Company Class B track) the rail was 75-lb., Am. Soc. C. E. section. The ties were mostly hewn soft wood ties, quite irregular in shape, but 20% were sawed ties, both oak and soft wood. The sawed ties were 6 by 8 in. by 8 ft., and the hewn ties were somewhat wider. The ties were spaced twenty to a rail length of 33 feet. The ballast was gravel about 36 in. deep. Due to the existence of the passing track (see Figs. 36 and 44) there was little slope on one side of the track. On the other the ballast sloped from 3 in. below the top of tie at its end at a 1:2 slope, forming a less effective support than was found on the other side. The

track surface was in indifferent condition, the track not having been tamped for some time. At places this track had been built over soft spots, which at times had given trouble by subsidence.

Fig. 37 gives the track-depression profiles at Locations *Q* and *P* of Class A track and Fig. 38 at Location *O*. In the diagrams load-depression diagrams at several points of these profiles are also given. The play in track denoted by the intercept at the vertical axis, found as shown in Fig. 30, was determined for a number of points along the rail; in Fig. 37 (and also in Figs. 39 and 40) a second set of profiles for one test has been drawn by plotting the net depressions found by deducting from the measured depression the play so determined. It will be noted that the profile of measured depressions is somewhat irregular and unsymmetrical. The profile of net depressions is more nearly regular and symmetrical, and the final load-depression ratio shows nearly the same stiffness throughout. This illustrates the variations in tie-bearing at points close together even in good track in fair condition; it has significance in relation to bearing pressure of tie, tamping conditions, and stress developed in the rail. The amount of the track play ranges from 0.02 to 0.17 in. in the track-depression profile of Figs. 37 and 38. In these tests, also, the depressions for one rail differed considerably from those for the other, though the net depressions were in close agreement.

Figs. 39 and 40 give the track-depression profiles on Class B track and also load-depression diagrams for several points along the rail for these tests. This track shows more irregularity along the length of the rail and more variation in play than was found in Class A track. At the point of maximum depression on the west rail of the upper profile of Fig. 39 the tie had been cut into by the tie-plate about $\frac{1}{2}$ in., a fact which was brought out when the load was applied.

In Table 14 the tie reactions are given for the several tests on the Chicago, Milwaukee and St. Paul Railway, the values being in terms of the load on one truck. The values at and between wheels range from 10 to 15 per cent. Class A with the heavier rail has a somewhat lower tie reaction than Class B.

The value of *u*, the modulus of elasticity of rail-support obtained from the track-depression profiles, is about 2 000 lb. per in. per in. for Class A track, and 900 for Class B track. The value for Class A track is materially higher than that found in the Illinois Central tests. Several factors may contribute to this—longer ties, broader ties (more bearing area in a rail length), gravel ballast instead of limestone ballast, greater depth of ballast, or better supporting conditions at ends of ties.

IV.—Depression, Flexure, and Bearing Pressure of Cross-Ties.

21.—The Cross-Tie and Its Action under Load.—In railroad track of ordinary construction the cross-tie performs many important func-

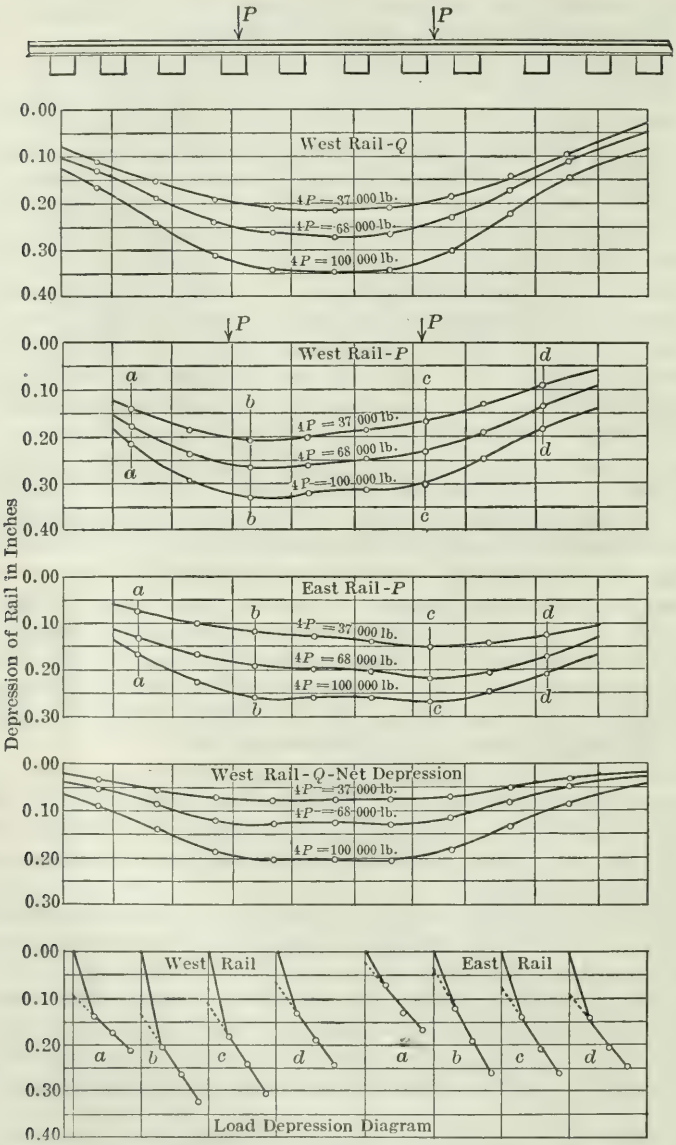


FIG. 37.—TRACK DEPRESSION PROFILES ON CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY: CLASS A TRACK.

tions—it bears the load transmitted from the rails, makes a definite connection and spacing between the two lines of rails, transmits the pressure to the bed of ballast and by its properties as a beam distributes this pressure along its length. It thus is subject to compression under and in the vicinity of the rail, to tension or compression as a structural connection between rails, and to beam action in giving distribution to bearing pressures along its bed. Of these the last bears the most important relation to the problems here considered; the others will not be treated at this time.

The cross-tie has many features which make it an acceptable part of ordinary track construction. It has great flexibility in meeting the

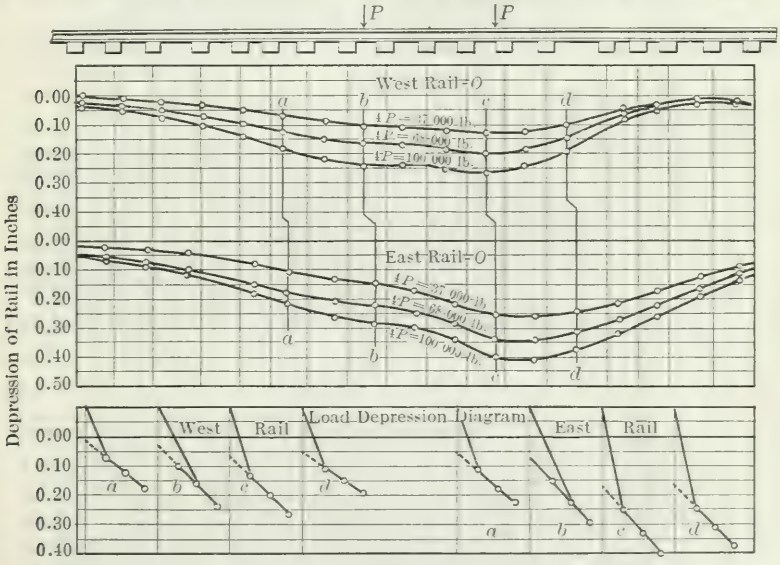


FIG. 38.—TRACK DEPRESSION PROFILES ON CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY: CLASS A TRACK.

varied and diverse requirements of both ordinary and unexpected track and load conditions. It gives facility in providing, maintaining, or shifting the gauge, line, and surface of track and in tie renewals and other changes in track, and is accepted as an economical and reliable element of ordinary track construction. It is known to be effective under very imperfect track conditions, being used where great variation of size and quality of tie, depth, and quality of ballast, and condition of tamping of track may be expected to cause marked differences in the requirements put upon it. Quantitative data on the flexural action of the tie and the way its loads are distributed along its bed, as well as of its depressions under load, therefore, should be of considerable value.

22.—Method of Attacking the Problem.—The problem to be considered relating to the action of the tie involves the distribution and intensity of the bearing pressure on its bed along its length and also the bending moments and stresses developed by this pressure and the load applied through the rails—these to be determined for the limits and ranges of values included in the variety of conditions to be found in track. This information will throw light on the question of the intensity of pressure on the ballast and also on the design of the tie, particularly if a tie of new form or dimensions or of materials other than wood were to be considered. The question of the intensity of pressure on the ballast for differing conditions of track tamping or of

TABLE 14.—TIE REACTIONS IN TERMS OF LOAD ON TRUCK. CHICAGO, MILWAUKEE & ST. PAUL RAILWAY.

Location of Tests	Wt. of Rail Lb. per Yd.	Size of Tie in Inches	Depth of Ballast	Load on Truck = 68 000 lb.									
O	100	7 x 8	36 in.	0.08	0.09	0.10	0.11	0.11	0.10	0.11	0.11	0.10	0.07
P	100	7 x 8	36 in.	0.04	0.08	0.11	0.13	0.13	0.13	0.12	0.12	0.09	0.05
Q	100	7 x 8	36 in.	0.03	0.07	0.11	0.15	0.15	0.15	0.14	0.11	0.07	0.02
R	75	6 x 8	36 in.	0.06	0.08	0.10	0.10	0.10	0.11	0.14	0.13	0.10	0.07
S	75	6 x 8	36 in.	0.06	0.09	0.13	0.14	0.13	0.13	0.12	0.10	0.06	0.03
				Load on Truck = 100 000 lb.									
O	100	7 x 8	36 in.	0.07	0.09	0.10	0.11	0.11	0.11	0.11	0.12	0.10	0.08
P	100	7 x 8	36 in.	0.04	0.08	0.11	0.13	0.13	0.13	0.13	0.11	0.09	0.05
Q	100	7 x 8	36 in.	0.04	0.07	0.11	0.14	0.14	0.14	0.13	0.11	0.08	0.03
R	75	7 x 8	36 in.	0.07	0.09	0.11	0.11	0.10	0.11	0.14	0.12	0.08	0.05
S	75	7 x 8	36 in.	0.06	0.09	0.13	0.14	0.13	0.13	0.12	0.10	0.06	0.03

track maintenance has a bearing on the questions relating to track ballasting. Even bearing pressures give good riding quality to the track, while spots with less and greater resistance to depression increase the stress in the rail and other parts of the structure and cause more rapid deterioration in the quality of the track. The determination of bending moments and stresses is closely related to various other parts of the problem.

A study of methods which may be used in attacking the problem of determining the distribution of the bearing pressure along the length of the tie will bring out the difficulties attending the experimental work. There is a great diversity of variations—differences in bearing pres-

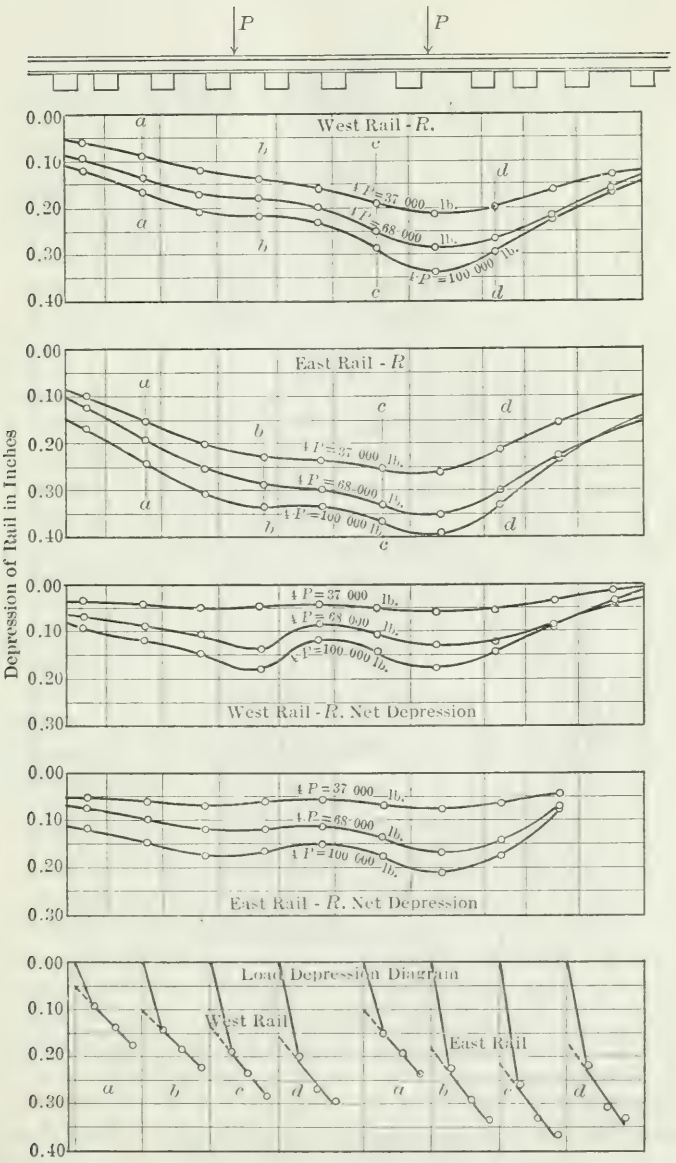


FIG. 39.—TRACK DEPRESSION PROFILES ON CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY: CLASS B TRACK.

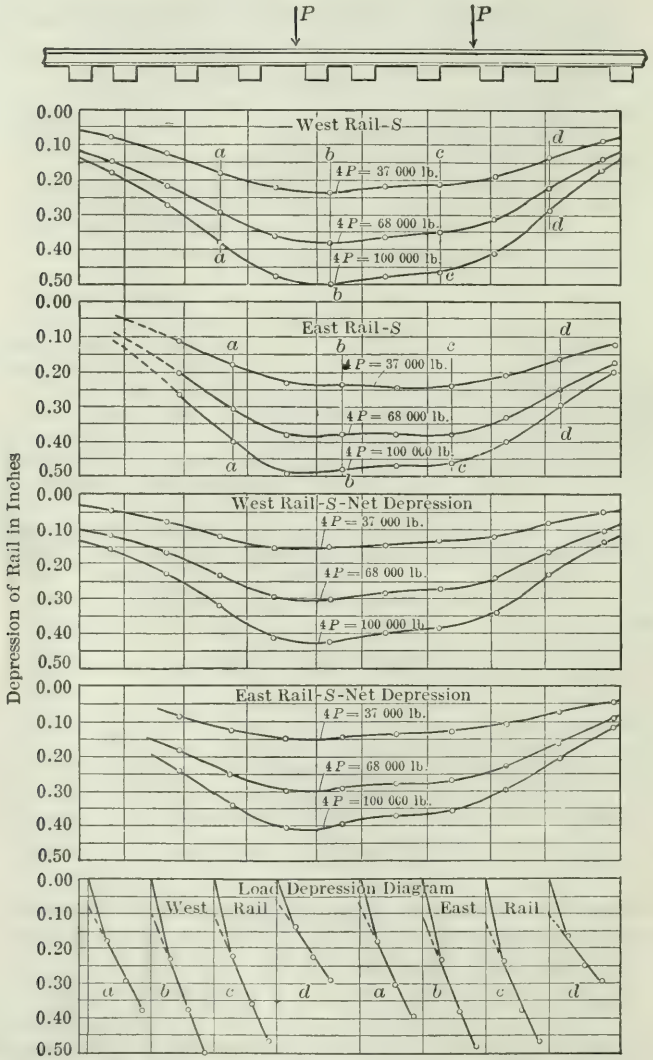


FIG. 40.—TRACK DEPRESSION PROFILES ON CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY: CLASS B TRACK.

sures across the width of the tie and along the tie; play between tie and its bed, which varies in different parts of the length of the tie and varies greatly with the nature of the tamping and the effects of the action of traffic; unequal stiffness (yieldability) of the ballast at various points—these illustrate difficulties encountered in the problem. The use of pressure-measuring devices between tie and ballast—such as the pressure capsule described in the first progress report—gave general information and served fairly well in the laboratory investigation; but the number required in the length of one tie to give the needed information and large number of ties required to determine the run of conditions, the disturbance of the track through inserting the instruments with the uncertainty as to whether the resulting condition of track was representative of normal track, and the chance for instruments becoming deranged in the interval elapsing until the track would seem to have reached normal conditions, all combined to make the direct measurement of bearing pressure impracticable as a method of getting the information desired on this subject. Of indirect methods the measurement of the strains or deformations along the length of the tie as produced by its bending was considered, but variations in the properties of the wood, especially under field conditions, as well as uncertainties of other kinds, rendered this method impracticable.

A trial was then made of a method of measuring the curve of flexure of the tie as it bends under the applied load in the track and making use of the relations between flexure, bending moment, and load as derived by mechanics of materials for a beam having the properties of the timber tie used. The measurements made included also the depression of the tie below its original position, as produced by the compression of the ballast and roadway or other conditions. This method is manifestly imperfect, having a number of limitations and uncertainties, but it is not difficult to use, the track conditions may be taken as they are found, and the number of tests may be chosen to cover a variety of conditions of ties and track. The measurement of tie depressions also gives information of value in the study of stresses in track. The method was used, therefore, as being the best known method applicable to the conditions of the track tests. It has given data of value, although certain uncertainties and inconsistencies are discernible in the results, especially those resulting from the local bearing compression in the vicinity of the rail. The details of the method will be described in Article 23. The consideration of the bending moments will be given in a succeeding article. Here it is only necessary to note that the curvature of the curve of flexure of the beam at any point is proportional to the corresponding moment and stress—the sharper the curve the greater the moment and the stress.

23.—Method of Making Tests.—In measuring the depression and flexure of the ties a stiff wooden reference bar (see Figs. 41 and 42) was placed under the rails between ties and close to the one on which measurements were taken. At each end of this bar was attached an iron cross-bar about 2 ft. long, the ends of which rested on two substantial stakes well driven into the ground, giving a firm support to the apparatus. These stakes were 7 ft. or more from the center of the track. Attached to the main bar were iron brackets (shown in Fig. 41) which projected over the tie. To each bracket was attached a piece of small angle iron; on this was bolted an Ames dial gauge which reacted against a tack driven at the mid-point of the width of the tie and served to measure the depression of the tie at that point. The arrangement of the attachment permitted adjustments vertically and horizontally.

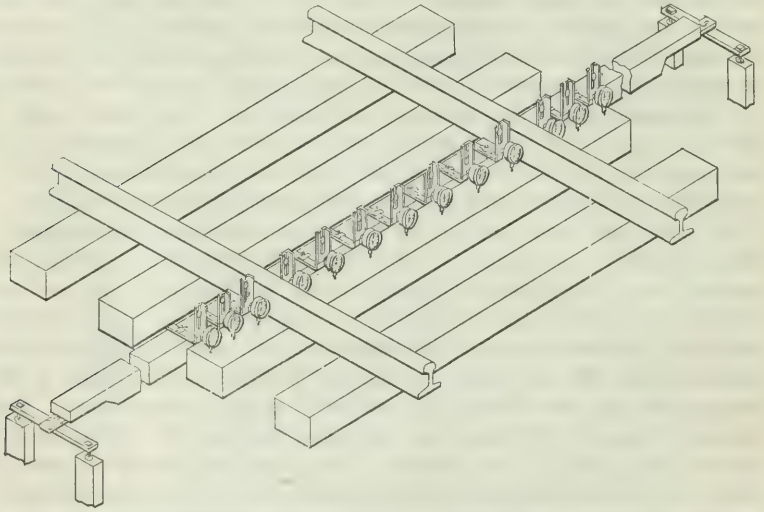


FIG. 41.—REFERENCE BAR FOR TEST OF DEPRESSION AND FLEXURE OF TIE.

Before applying a load the dials were adjusted to read zero. For each load applied the dials were read to the nearest half-thousandth of an inch. When the load was removed, a small depression was indicated by the dials and they were again set at zero. The same load was again applied, the process being continued until three consistent sets of readings were obtained. The average of these sets of readings was used.

In the tests on the Illinois Central Railroad nine dial gauges were used—five between rails and two on the outer side of each rail. This number of points was found not to determine the curve of flexure as accurately as was desired, and in the later tests made on the Chicago, Milwaukee and St. Paul Railway the number of dials was increased to thirteen, seven between rails and three on the outer side of each rail; this

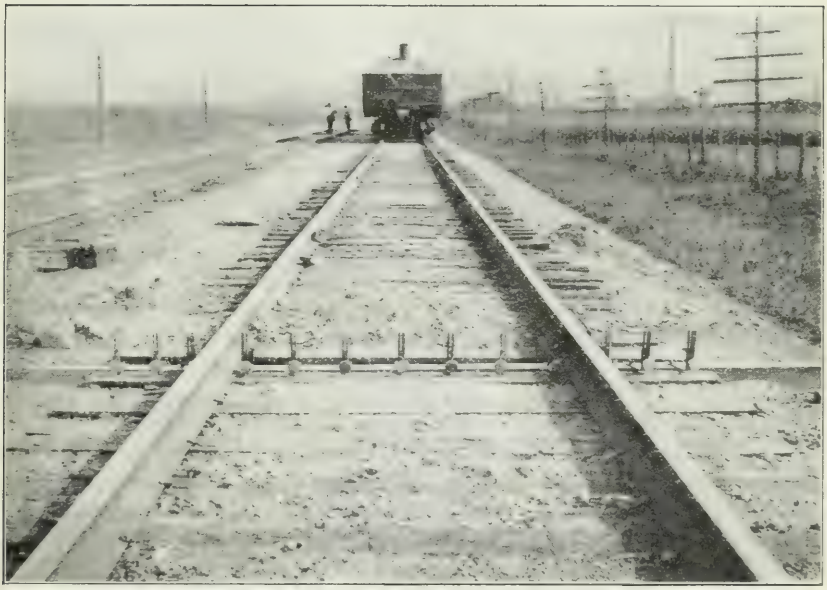


FIG. 42.—REFERENCE BAR FOR TEST OF DEPRESSION AND FLEXURE OF TIE.

improved the character of the data very much. In each case the dials next the rails were 1 to 2 in. away from the base.

The loads were given by means of loaded cars, as already described for the tests of track depression. The three loads on a single truck (two axles) were 18 000, 53 000, and 100 000 lb. on the Illinois Central Railroad and 37 000, 68 000, and 100 000 lb. on the Chicago, Milwaukee and St. Paul Railway. The leading wheels of the truck carrying the load were placed directly over the tie under test. The amount of the load considered to bear on the tie under test was taken from the results of the track-depression tests at the given location. The smallest load used was generally more than sufficient to overcome the play between tie and ballast, though of course this should not be taken to mean that under this condition the pressure on the bed of ballast is distributed uniformly along the length of the tie. The largest load used gave a tie reaction considerably less than that which would be caused at and between drivers by an ordinary Mikado or Santa Fe type locomotive. In the study of the data the largest load was generally used, though the smaller loads have given valuable information on the action of the tie.

The readings in all the tests checked very closely and the results are concordant. The effect of the rail bearing in compressing the fibers adjacent to the rail is unknown, but the reading of the dials next to the rail must include the general depression and bending of the tie and also a third element—the influence of the local downward compression of the tie due to the bearing pressure of the rail; the latter will have to be taken into account in interpreting the curves of flexure as related to bending moments and stresses.

The form of reference bar given in Fig. 41 is the improved form used in the tests on the Chicago, Milwaukee and St. Paul Railway. The bar used in the earlier tests was lighter and less stiff. In the use of this apparatus it should be noted that it was necessary to remove the ballast between the tie to be tested and an adjoining tie to a depth of 4 or 5 in. below the top of tie. This probably had little effect upon the action of the tie during the tests, although under continued traffic of no greater loads the effect might be noticeable.

24.—Location and Description of Track.—Fig. 36 gives data on the location of ties at which measurements of depression and flexure were made, together with condition of the track. The letters *B*, *C*, and *K* designating location on the Illinois Central Railroad refer to the sections of track north of Champaign, Ill., shown in Fig. 26 of the first progress report. The ties on the main-line track were the sawed oak ties (6 by 8 in. by 8 ft.) furnished for the test track in 1914. The tests on the freight lead were made on two hewn oak ties about 6½ by 8¾ in. by 8 ft. and a sawed oak tie 6 by 8 in. by 8 ft. The location of the tests on the Chicago, Milwaukee and St. Paul Railway has already been described under "Track Depression Tests on the Chicago, Milwaukee

and St. Paul Railway". Sawed ties were selected for test because of their uniformity of cross-section. The ties on Class A track were 7 by 8 in. by 8 ft. 6 in.; those on Class B track 6 by 8 in. by 8 ft. Tie-plates 8 by 9 in. were on all ties in the Class B track.

Attention should be called to the condition of the cross-section of track at the test locations given in Figs. 43 and 44. In some of the sections on the Illinois Central Railroad it may be seen that there is a scant amount of ballast above the bottom of the tie near its ends, particularly on the outer side of the track. On the Chicago, Milwaukee and St. Paul Railway, as has already been pointed out, the sections are such that there was opportunity for full support at the end of the tie. The condition of the track surface has been described under "Track Depressions"; it may be well again to call attention to the fact that the track in all the test locations had not received a general tamping for some time and that in some locations it was apparent that the track was in indifferent condition as to surface and tamping, and thus furnished opportunity to determine depressions and flexure under other than best conditions, and really under conditions frequently found in practice.

25.—Diagrams of Depression and Flexure and Their Significance.— Figs. 45 to 54 give the depressions and flexure of the individual ties at a truck load of 100 000 lb. The load on a single tie, as determined by the track-depression profiles, ranged from 11 000 to 18 000 lb., perhaps averaging 15 000 lb. The vertical scale is relatively large, thus bringing out the curvature, which of course is slight, as is the case with any short stiff beam. The tie depression under this load varies from 0.35 to 0.5 in. Figs. 55 and 56 give composite diagrams of the flexure of several ties at the same location and which have the same general characteristics of flexure; the differences in depressions and the tilting of the ties are not here considered—the diagrams are plotted with reference to a line drawn from rail to rail. In Fig. 47 curves are drawn to represent the depressions and flexure for all three truck loads. The dotted lines given in this diagram (and in the other diagrams), obtained by a method described subsequently, are intended to represent the position of the ballast bed under the tie when there is no load on the tie; the dotted lines given are for the tie "before tamping".

An examination of the diagrams of depression and flexure brings out a variety of conditions. It will be observed that there is bending of tie in every case, though the form of the curve varies greatly. It is evident that there must be great differences in the way in which the bearing pressure varies along the length of the tie under the conditions to which ties are subjected. In many ties there is play between the tie and its bed at one point or another or even over a considerable portion of its length, which not only gives unevenness of track depression but increases the intensity of bearing pressure at some point along the tie

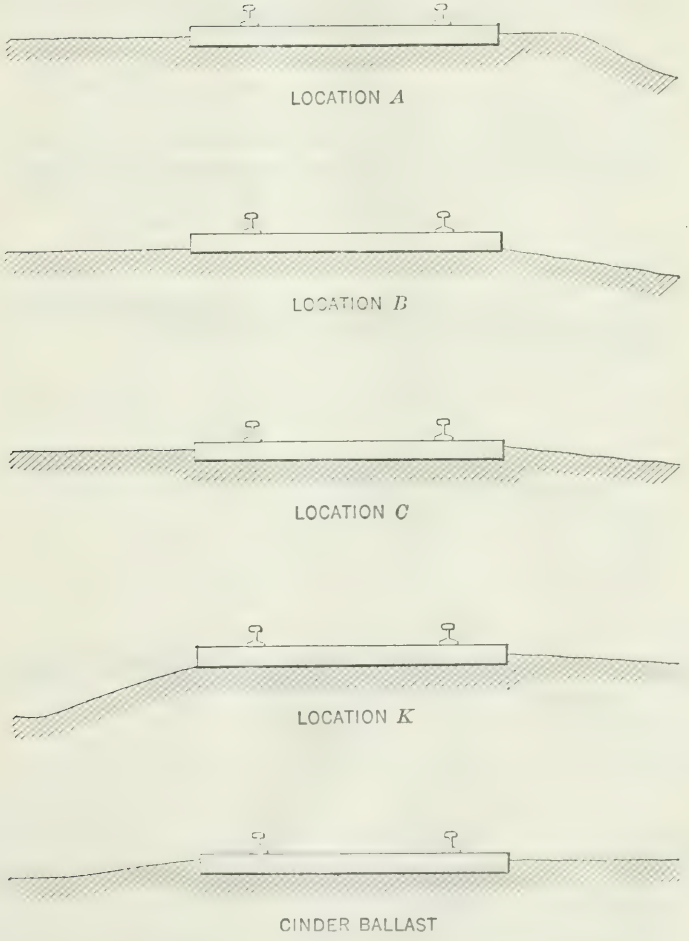


FIG. 43.—CROSS-SECTIONS OF TRACK ON ILLINOIS CENTRAL RAILROAD.

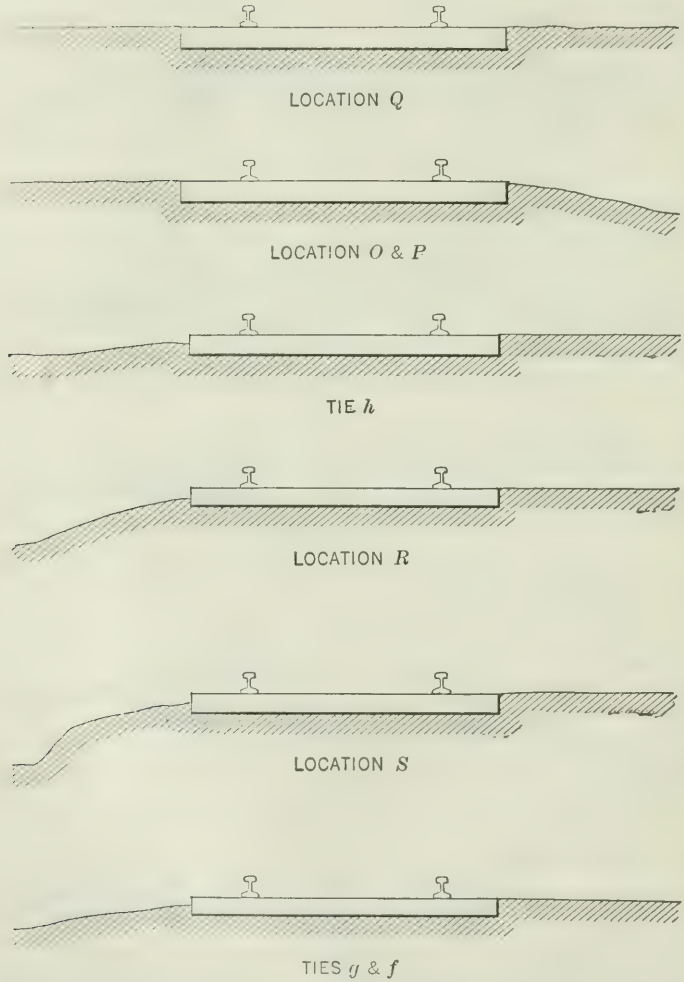


FIG. 44.—CROSS-SECTIONS OF TRACK ON CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

and gives added bending stresses in the tie. It is clear that there is a great variety of distribution of bending moments along the tie—the bending moment at a section at the middle of the tie may be positive in one tie and negative in another, as is shown for Tie *o* in Fig. 50 for "after tamping" and "before tamping". Attention should be called to the fact that the tamping done was slight and at the ends of the tie only; the term "after tamping" should not be taken to mean a general tamping.

The conditions just referred to are ordinarily termed endbound and centerbound. It will appear that the latter condition is very common; in fact, the upward curvature at the middle and a large negative

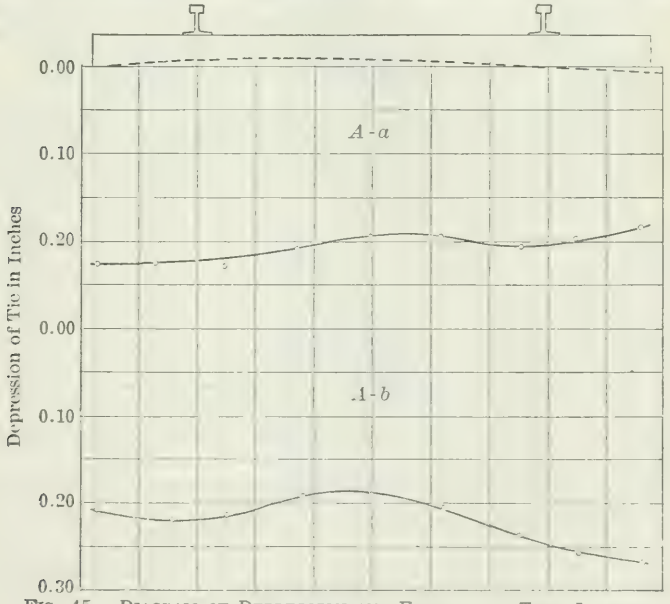


FIG. 45.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

moment accompanying it may be expected in most track after traffic has had its effect upon track condition. The immediate effect of tamping a tie in ballast already well compacted being to change the bearing conditions along the tie (the compacting of the ballast in the tamping having only a secondary effect), a fresh tamping or a change in the place of tamping makes a marked effect on the flexure of the tie. The ties in Location *B* were found to be loose at their ends under the conditions of the track in service, with a resulting flexure curve shown in Figs. 46, 47, and 48 for Ties *c* to *k*. The part of these ties outside the rails was then tamped slightly, not vigorously, and the curve in Fig. 49 for Tie *g* slightly tamped shows that the bearing is largely at the

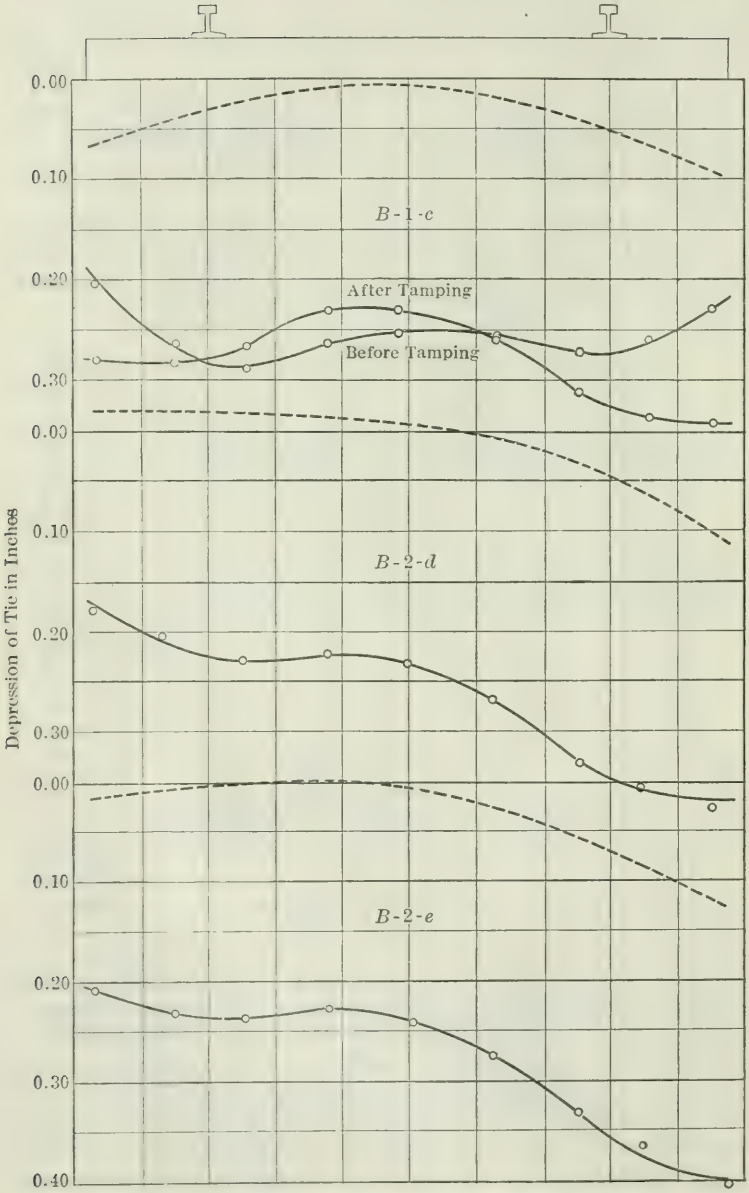


FIG. 46.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

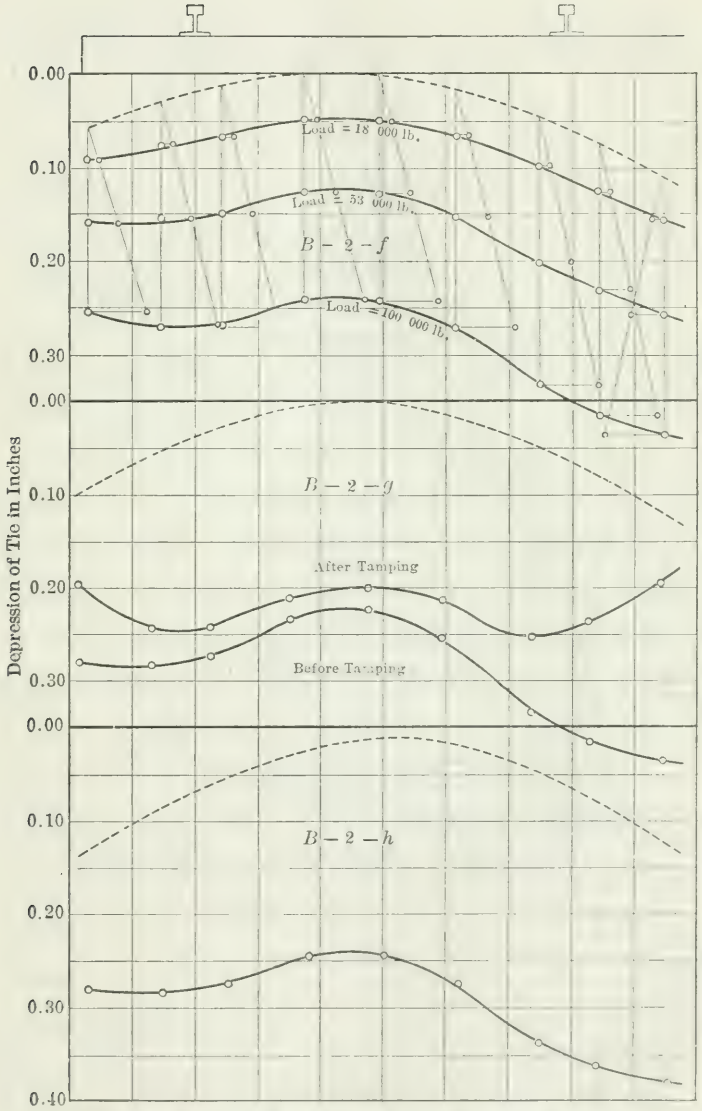


FIG. 47.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

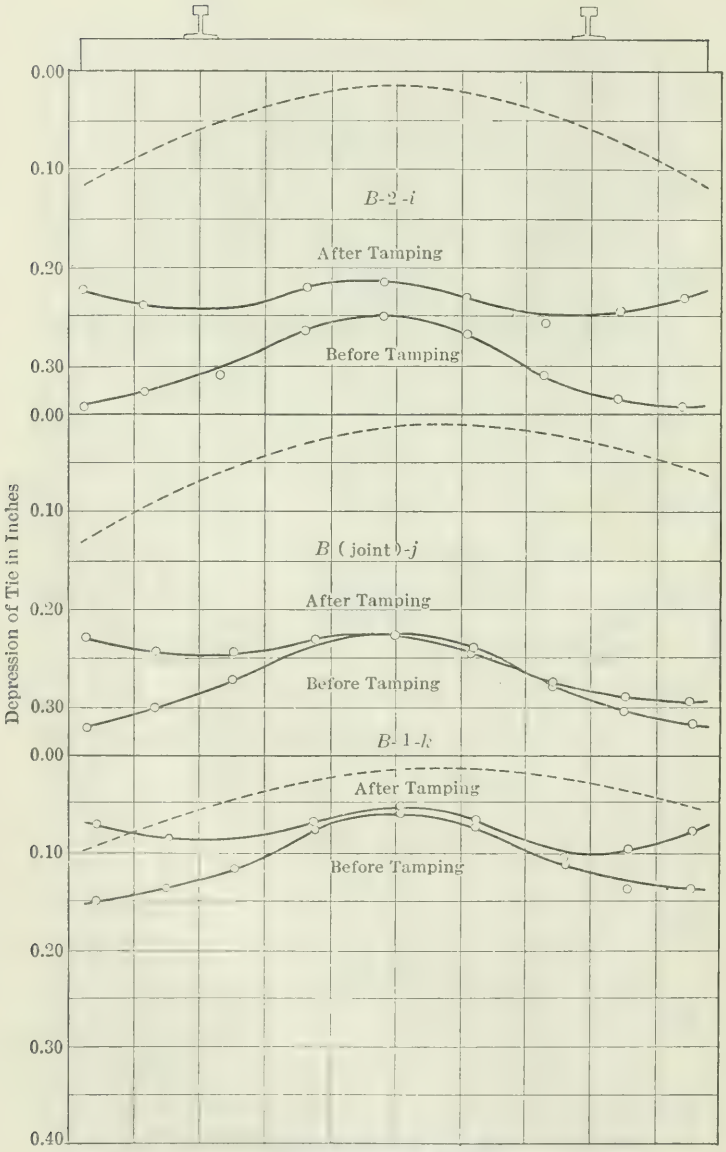


FIG. 48.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

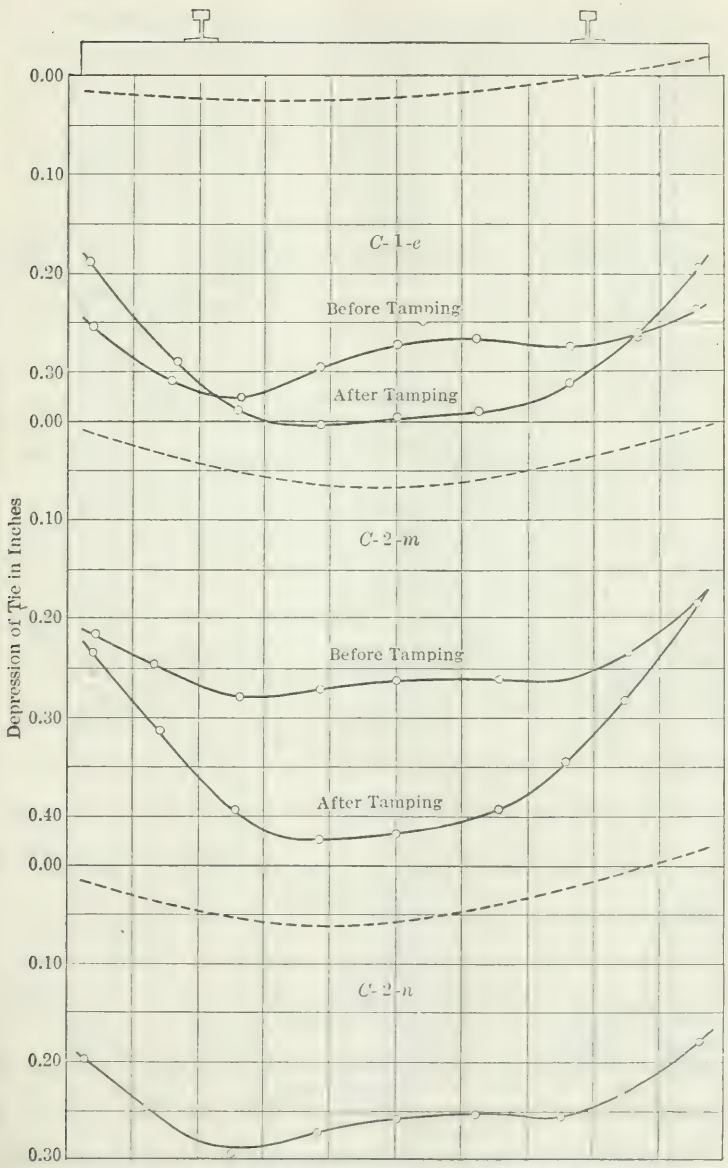


FIG. 49.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

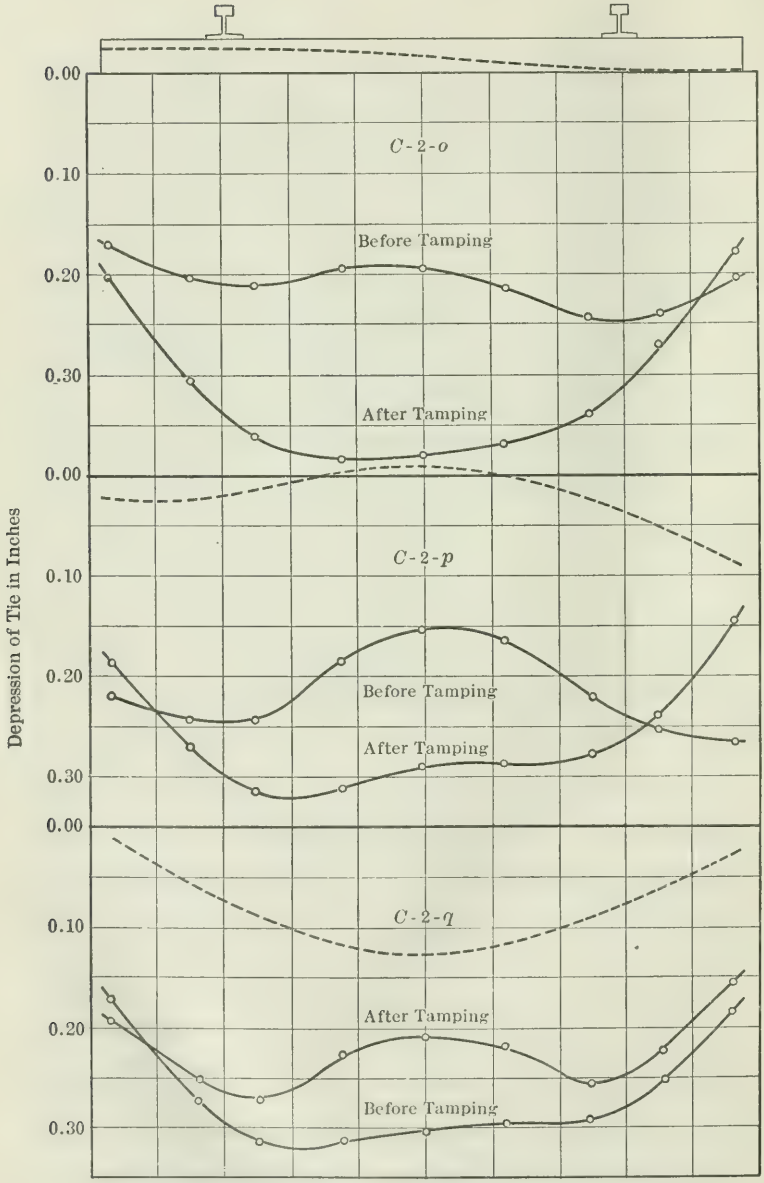


FIG 50.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

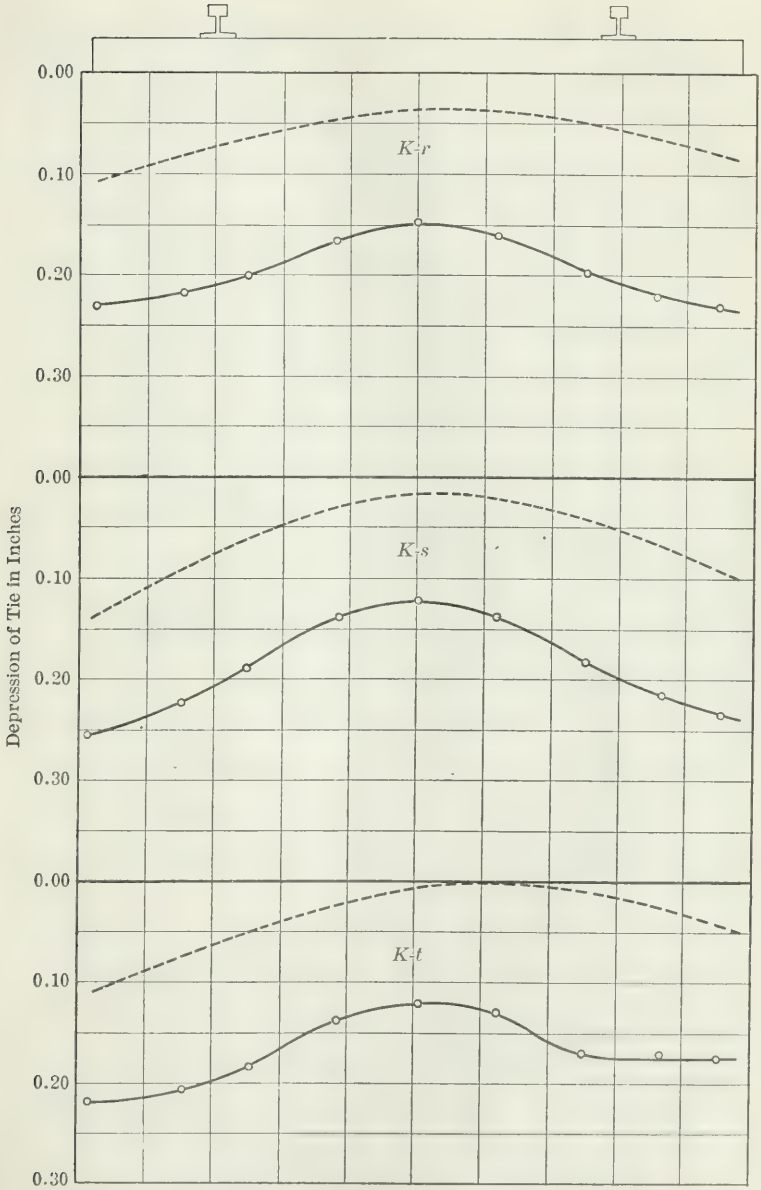


FIG. 51.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

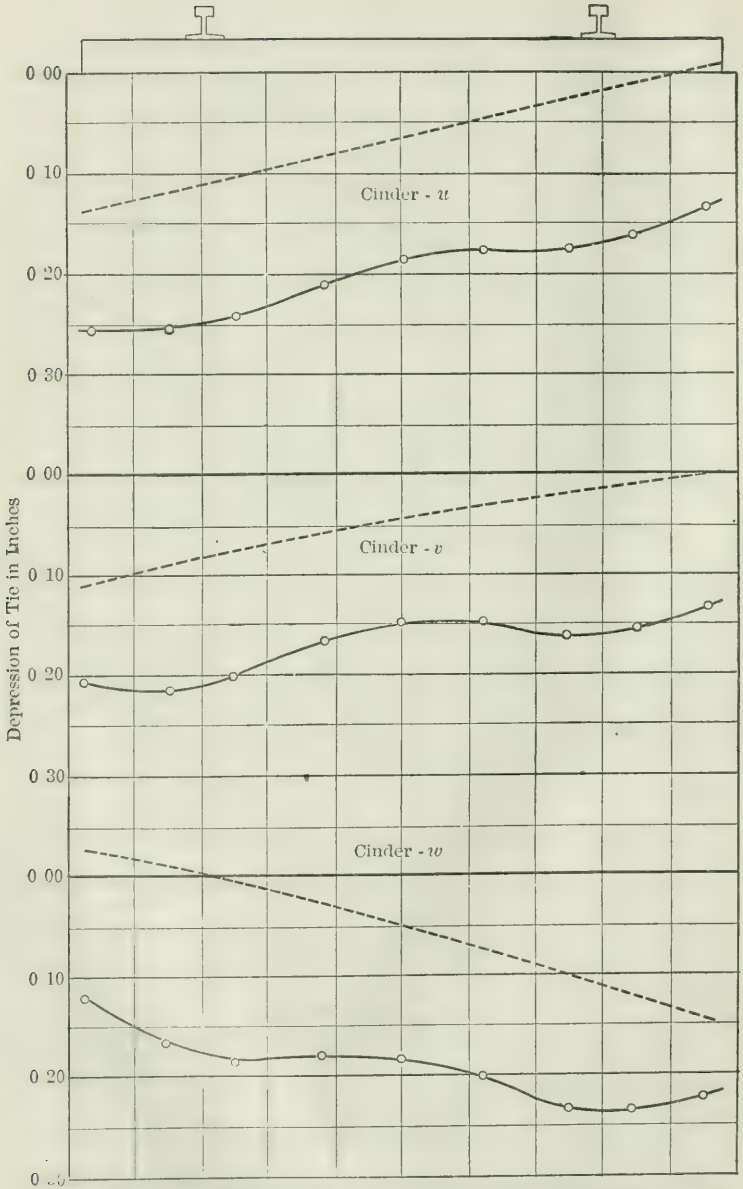


FIG. 52.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, ILLINOIS CENTRAL RAILROAD.

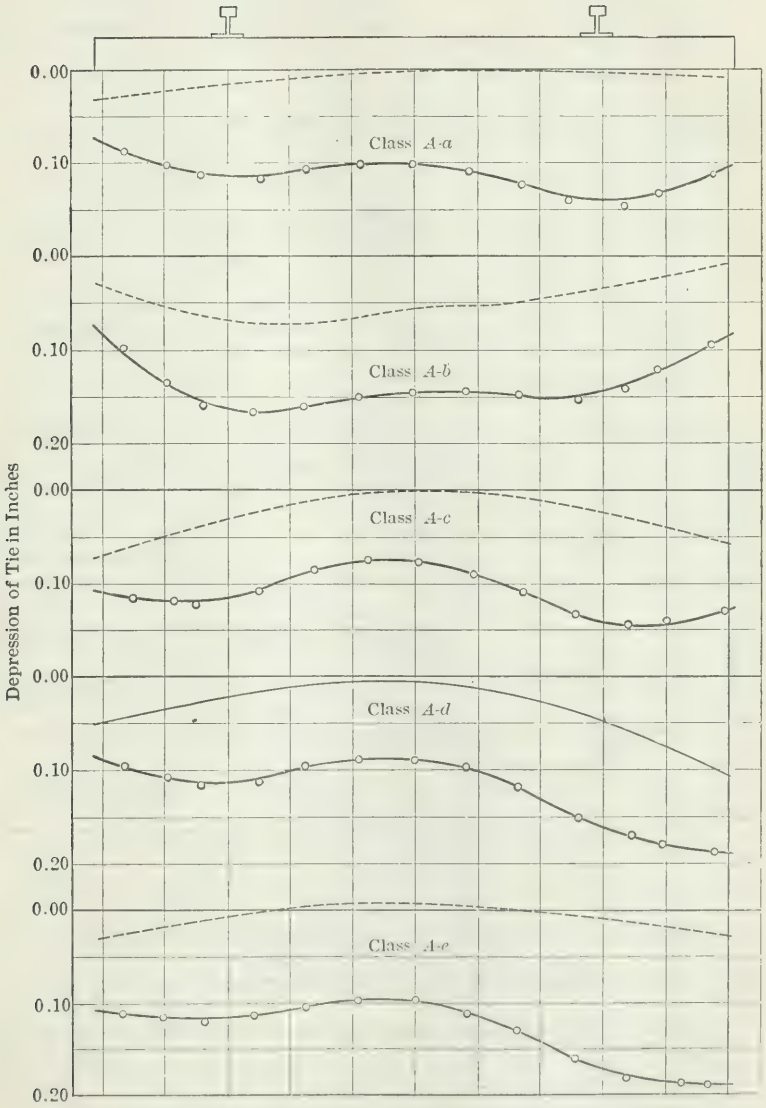


FIG. 53.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

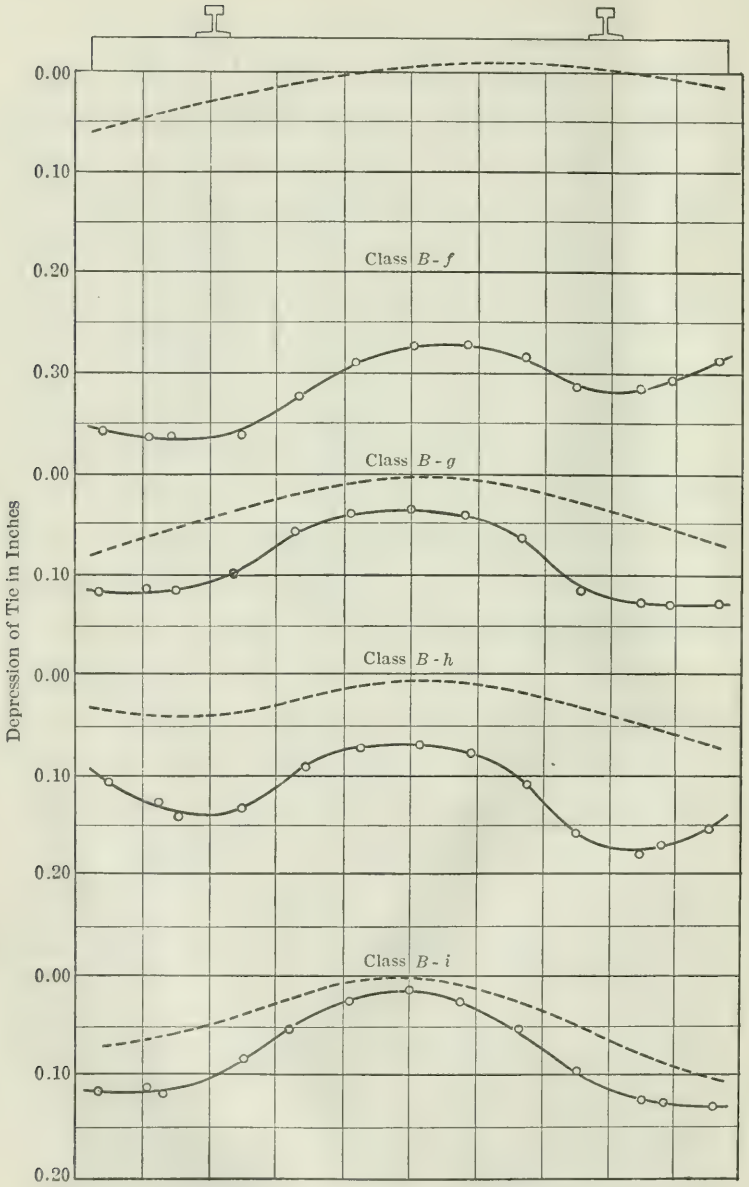


FIG. 54.—DIAGRAM OF DEPRESSION AND FLEXURE OF TIES, CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

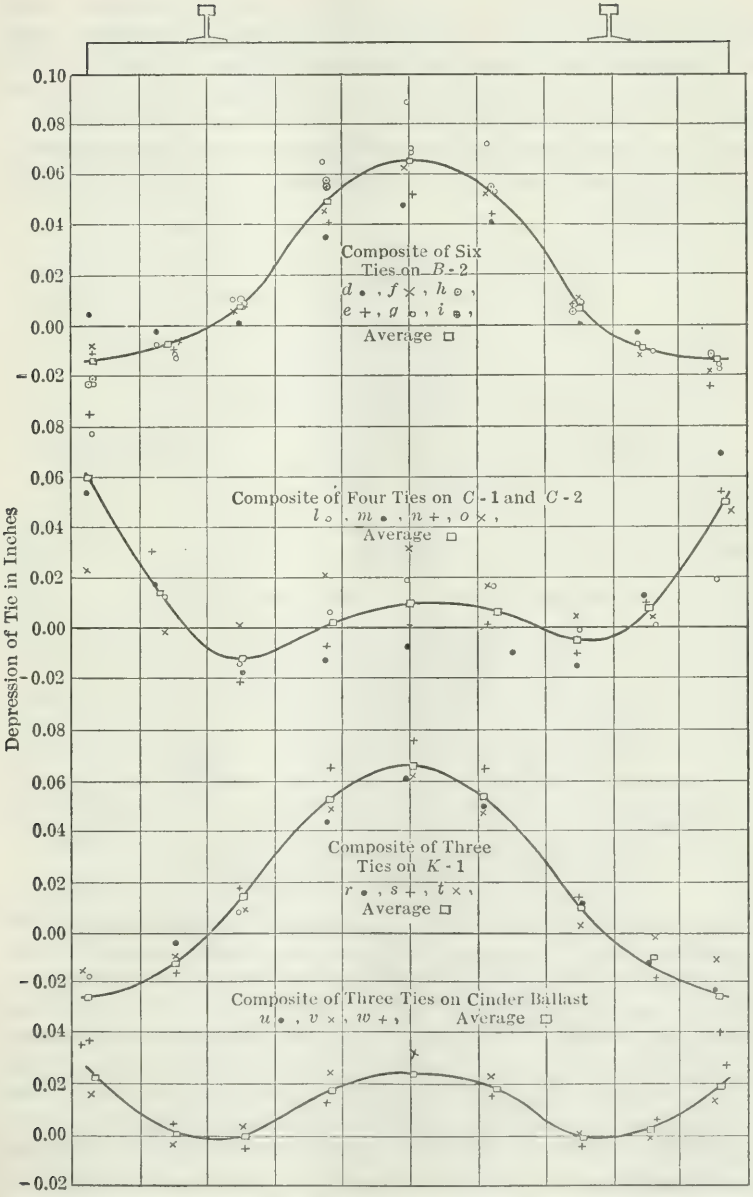

Depression of Tie in Inches
0.10
0.08
0.06
0.04
0.02
0.00
-0.02
Composite of Six Ties on B-2
d •, f ×, h ⊙,
e +, g ○, i ⊕,
Average □
Composite of Four Ties on C-1 and C-2
l ○, m •, n +, o ×,
Average □
Composite of Three Ties on K-1
r •, s +, t ×,
Average □
Composite of Three Ties on Cinder Ballast
u •, v ×, w +, Average □

FIG. 55.—COMPOSITE DIAGRAMS OF TIE FLEXURE, ILLINOIS CENTRAL RAILROAD.

ends of the tie. Change in position of the ballast under traffic conditions will soon modify the distribution of bearing pressure. Even though the tamping is restricted to a space adjacent to the two rails, the depression of the track under load may be expected to bring some bearing pressure at the middle of the tie length, and the action of a repetition of applications of pressures of high intensity over the tamped portion will cause a movement of the ballast and a redistribution of bearing pressures.

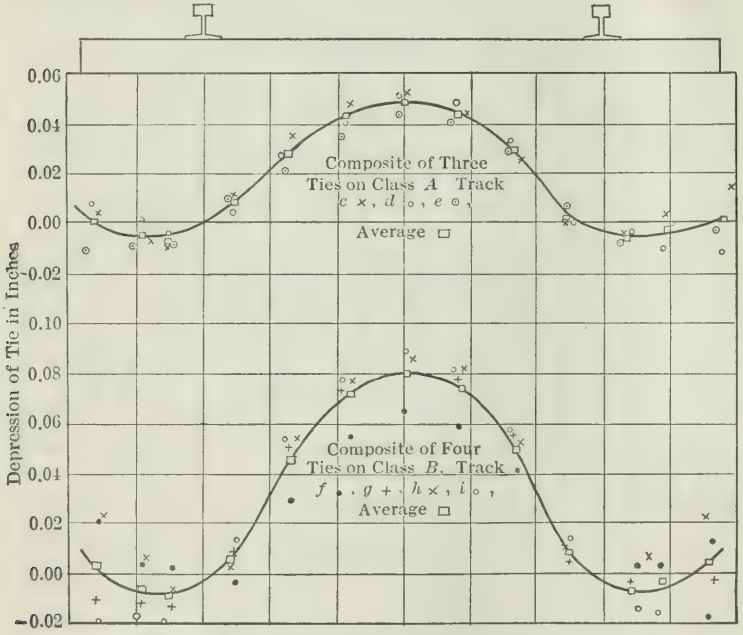


FIG. 56.—COMPOSITE DIAGRAMS OF TIE FLEXURE, CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The condition of the ballast section at and beyond the end of the tie seems to have a modifying influence upon bearing conditions. The scantness of the shoulder in Section *B* of the Illinois Central Railroad evidently contributed to the loose ends and play of many of these ties, especially those the looseness of which was noticeable by inspection; the effect of this probably becoming apparent only after traffic had produced appreciable changes in the position of the particles of ballast. The more complete support at end of tie in the ballast section on the Chicago, Milwaukee and St. Paul Railway gives conditions which are quite different.

The initial position of the ballast bed with respect to the bottom of the tie for the compact condition of ballast existing in the track tested

TABLE 15.—Tests of Depression and Flexure of Ties..

Location.	Number tested.	Designation.	Ballast.	Remarks.
Illinois Central Railroad				
A	2	*A a, b*	12-in. stone..	85-lb. rail; 7 by 9 in. by 8-ft. oak ties.
B	1	*B*-1 *c*	do.	6 by 8-in. by 8-ft. oak ties.
	6	*B*-2 *d, e, f, g, h, i*	do.	do.
B (joint)	2	*B* (joint) *j, k*	do.	do.
C	1	*C*-1 *l*	6-in. stone...	do.
	5	*C*-2 *m, n, o, p, q*	do.	do.
K	3	*K r, s, t*	24-in. stone..	125-lb. rail; 6 by 8-in. by 8-ft. oak ties.
North-bound freight lead.	3	Cinder, *u, r, w*	12-in. cinder.	85-lb. rail; hewn ties.
Chicago, Milwaukee and St. Paul Railway				
Class A track	5	Class A *a, b, c, d, e*	36-in. gravel.	100-lb. rail; 7 by 8 in. by 8-ft. 6-in. oak ties.
Class B track	4	Class B *f, g, h, i*	do.	75-lb. rail; 6 by 8-in. by 8-ft. soft wood ties.

may be found approximately by the following method. In any diagram of depression and flexure (as, for example, Fig. 47), for each point along the tie and each depressed position, lay off a horizontal distance proportional to the load producing the given tie depression. The points so measured for the three loads used will be found to lie almost in a straight line. Produce such a straight line to intersect the vertical line through the initial position of the point on the tie. This intersection would be the initial position of the ballast bed under the point on the tie if the bearing pressure was uniform along the tie. A second approximation (which it is not generally necessary to make) may be made by assuming that at any given point the intensity of the bearing pressure accompanying each load varies as the vertical distance from the point just found to the position occupied at the given load. This method will locate quite closely the initial position of the bed for track with well compacted ballast. The bending of the tie to fit minor depressions, such as that which may exist directly under the rail, is not taken into account; this would modify the results but slightly. The dotted lines given in the diagrams, obtained in the manner described, may then be expected to represent fairly closely the initial position of the bed under the tie. Play is given by the space between this line and the tie. It will be seen from the diagrams that the ties which show large upward curvature at the middle have their beds curved in

the same manner; in other cases the bed at the middle is much lower than at the ends of the tie—the play between tie and bed being at the ends in one case and at the middle in the other.

As the amount of depression affects the modulus of elasticity of rail-support, u, it may be expected that the condition of tie and ballast will be influential factors. In determining u, the play in track depression was omitted from the values, though of course it enters into the development of stresses in the rail and is known to produce higher stresses than would otherwise occur, especially if the play varies from tie to tie. The bending of the ties, however, and especially that under the rail, tends to make low values of u. Another element contributing toward the higher value of u in the Chicago, Milwaukee and St. Paul Railway track, already referred to, seems to be the gravel ballast; there are fewer voids and a greater cross-section of material to take the load and therefore the amount of compression in the ballast may be expected to be less. The modulus of elasticity of the limestone used on the Illinois Central Railroad is less than that of the gravel of the Chicago, Milwaukee and St. Paul Railway.

Attention is called to the fact that the deflections in the flexure curve of the 7 by 9-in. ties are much smaller than those in the 6 by 8-in. ties; for the same loading the deflections may be expected to vary inversely as the moment of inertia of the section and the moment of inertia of the 7 by 9-in. tie is nearly 70% greater than that of the 6 by 8-in. tie. It may be added that the greater stiffness of tie should help in making more uniform pressures along the length of the tie.

The joint ties, j and k (see Figs. 36 and 48) were chosen for test because they were particularly loose and showed much play. The effect of tamping on this tie is very noticeable. The condition of the joint Tie i (see Fig. 48) is not especially different from the other ties in the vicinity.

The composite diagram for the ties on cinder ballast gives a good appearance, symmetrical and with apparent good end bearing, but it should be noted that these curves are not plotted from gross depressions; the individual ties give markedly different curves, tilting first to one side of the track and then to the other. This is due not only to the ballast and condition of tamping, but to the fact that the ties were unevenly spaced and did not always lie at right angles to the rails. The change from the greatest depression occurring at the end of a tie at one side of the track to the greatest depression at the other side of the track within a short distance was not confined to the freight tracks and cinder ballast. The tilting of the tie to one side of the track, however, was more common where the shoulder of ballast was scant.

The effect of the greater length of tie in Class A track will be considered in Article 26.

26.—Bearing Pressure and Bending Moment.—If the intensity of the tie reaction were directly proportional to the measured tie depression it would be easy to find the distribution of the tie reaction along the length of the tie by the use of the diagrams of tie depressions already given. Two variations from this law of distribution of pressure are known to exist; (*a*), due to the condition of ballast bed under the tie the unloaded tie may not touch its bed at all points, and (*b*), the ballast may be more compact and stiffer at one point than at another. Most of the uncertainty due to the first condition may be overcome by the use of the dotted line in the diagrams of tie depression as expressing the initial position of the tie bed; there may be a further depression in the bed due to the bending of the tie immediately under the rail, as shown in Fig. 54 (left end of Tie *h*). The dotted lines of the diagrams generally indicate a centerbound condition; this gives a bending moment at the middle of the tie when it is brought to a full bearing and increases the bending moment at all loads and also results in an increase of intensity of bearing pressure at the middle. A permanently formed depression at the rail acts to relieve the intensity of tie reaction there but increases the bending moment at this point when a load is applied which brings the tie down to full bearing. The second condition, that not giving direct proportionality between depression and intensity of bearing pressure, is particularly evident in freshly placed ballast. In well compacted ballast the variation from proportionality may be expected to be slight. Immediately after tamping, the bed may be somewhat stiffer at the point of tamping; as the effect of tamping disappears through the movement of the particles of ballast, the yieldability of the ballast under load, that is, the modulus of elasticity of the mass, tends to become more uniform along the length of the tie. By taking into consideration the effect of variations in these conditions the diagrams of tie depression and flexure will be found useful in indicating the range of variation of bearing pressure and bending moment.

It will be of interest to consider a variety of assumed distributions of bearing pressure (tie reactions) and the resulting distribution of bending moments. Fig. 57 gives a number of hypothetical cases. Uniform pressure shown at (*a*) cannot be expected. For a few inches at the end of the tie the bearing pressure must be small. The flexure of the tie itself will also produce a variation from uniformity of pressure. At (*b*) is shown a diagram indicating the influence of these factors when the initial bearing is fair and the stiffness of the bed is uniform along the length of the tie. At (*c*) the distribution is sometimes thought to exist when the tie is tamped for a foot or so on each side of the rail. Diagram (*d*) gives the distribution for freshly tamped sand ballast as found by laboratory measurements with pressure capsules—little pressure was here found at the middle of the tie; (*e*) assumes the

principal bearing to be at the rails tapering off in each direction; (*f*) assumes maximum intensity at the end of the tie and (*g*) at the middle; (*h*) represents a form of distribution for the centerbound tie, somewhat as was found for the medium load in some of the tests; (*i*) represents the distribution for an endbound tie; (*j*) may be considered as representative of the pressures under a well tamped tie in compact ballast after traffic has begun to have an effect. It is possible that at the point of greatest depression, as immediately under the rail, a permanent depression will be formed and the tie will not touch the bed until it has bent considerably at this point. This condition tends to give a decreased intensity under the rail, as shown at (*k*), but the effect of this on the distribution is not great.

TABLE 16.—Bending Moments in Tie for Assumed Distributions of Bearing Pressure.

W = Total load, in pounds, carried by one tie. Moment is then given in pound-inches.

Distribution as designated in Fig. 57.	8-Ft. Tie.		8-Ft. 6-In. Tie.		9-Ft. Tie.	
	At rail.	At middle.	At rail.	At middle.	At rail.	At middle.
a	$+1.5\ W$	$-2.7\ W$	$+1.9\ W$	$-2.0\ W$	$+2.5\ W$	$-1.2\ W$
b	$+1.6\ W$	$-1.7\ W$	$+2.1\ W$	$-1.0\ W$	$+2.5\ W$	$-0.5\ W$
c	$+1.8\ W$	$0.0\ W$	$+1.8\ W$	$0.0\ W$	$+1.8\ W$	$0.0\ W$
d	$+1.3\ W$	$-1.5\ W$	$+1.8\ W$	$-0.9\ W$	$+2.1\ W$	$-0.5\ W$
e	$+0.9\ W$	$-1.8\ W$	$+1.2\ W$	$-1.3\ W$	$+1.5\ W$	$-0.8\ W$
f	$+2.8\ W$	$+1.2\ W$	$+3.6\ W$	$+2.2\ W$	$+4.4\ W$	$+3.2\ W$
g	$+0.1\ W$	$-6.7\ W$	$+0.3\ W$	$-6.2\ W$	$+0.5\ W$	$-5.7\ W$
h	$+1.3\ W$	$-3.9\ W$	$+1.6\ W$	$-3.3\ W$	$+2.0\ W$	$-2.6\ W$
i	$+2.0\ W$	$-0.4\ W$	$+2.7\ W$	$+0.6\ W$	$+3.3\ W$	$+1.4\ W$
j	$+1.4\ W$	$-2.4\ W$	$+1.9\ W$	$-1.8\ W$	$+2.4\ W$	$-1.0\ W$
k	$+1.6\ W$	$-1.9\ W$	$+2.1\ W$	$-1.1\ W$	$+2.6\ W$	$-0.4\ W$
l	$+4.9\ W$	$+5.2\ W$	$+5.7\ W$	$+6.0\ W$	$+6.4\ W$	$+6.7\ W$
m	$+2.4\ W$	$+0.6\ W$	$+2.1\ W$	$+1.4\ W$	$+3.6\ W$	$+2.2\ W$

The relative magnitudes of the bending moments at points along the tie are shown for these cases by the diagrams at the right in Fig. 57. Table 16 gives, for the distributions of bearing pressure assumed in Fig. 57, the values of the bending moment at two sections of the tie and for three lengths of tie. W is the total load on the tie in pounds and the moment is expressed in pound-inches. It is seen that there is a considerable range of values of the moments, both at the middle of the tie and at the rail. It is also to be noted that a slight change in the distribution of the intensity of bearing pressure along the tie makes a relatively large change in the amount and distribution of the bending moments.

A study of the flexure curves of the ties tested in track and a comparison with flexure curves corresponding to the distribution of moments given in Fig. 57 show that a variety of conditions of tie

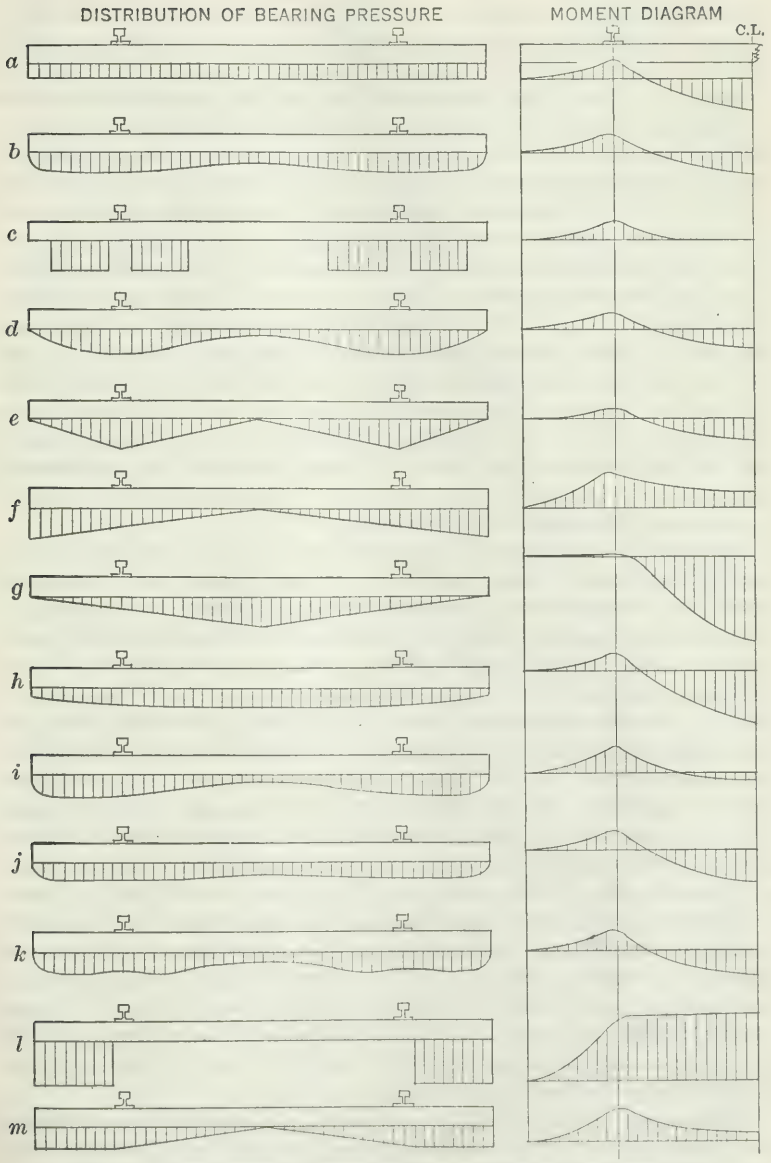


FIG. 57.—HYPOTHETICAL DISTRIBUTIONS OF BEARING PRESSURE AND CORRESPONDING MOMENT DIAGRAMS FOR TIE 8 FT. LONG.

bearing may exist. Even in well-tamped track the uniform distribution of bearing pressure indicated at (*a*) cannot occur. The intensity of bearing pressure will be greater near the rail than at the middle or end of the tie since the bending of the tie will result in larger depressions at that point. The more flexible the tie, the greater the variation in intensity; likewise, the stiffer the ballast, the greater the variation—all resulting in increased intensity of bearing pressure at the rail. Such a condition gives a smaller value of the bending moment at the middle of the tie than would be found for uniform bearing pressure. If the ballast at the middle of the track is less compact than elsewhere the difference becomes more pronounced. The bending moment at the rail for this condition is less than for the endbound condition. The condition given at (*c*) in Fig. 57 may be expected to merge into that at (*d*). With a concentration of pressures near the rails it may be expected that a permanent depression will soon be formed under the rail, a greater depression all along the tie will then occur upon the application of the heavy loads, and the bearing pressures will be distributed more uniformly along the tie. A permanently formed depression under the rail will increase the bending moment in the tie at the section under the rail.

In centerbound track, for the smaller loads, the tie reactions are carried mostly in a short space at the middle of the tie. These small loads act to bend the tie downward at its middle to fit the shape of the bed (as given for Tie *i* by the dotted line in Fig. 48); only a small load is needed to give this bending, particularly as the bed at the middle of the tie will be depressed under the action of the bearing pressure. Under the conditions of the tests with centerbound track, the load required to bring the tie to a bearing under the rail was probably not greater than 10% of the 100 000-lb. load used. The negative moment produced by this small load is, of course, relatively large. The loads used in the test give full bearing along the tie, the depressions from the original position of the bed (dotted line in the diagram) not differing greatly from what would be found in well-tamped track.

For an endbound tie (see Tie *q* in Fig. 50), the bending moments developed with the small loads are all positive (curvature all downward), and for unusual conditions they may remain positive at the higher loads, as is instanced in Tie *o* after tamping (see Fig. 50). A more usual condition for what is commonly included as an endbound condition develops positive moments at the middle of the tie for the smaller loads and negative moments for the larger loads.

The curves of flexure of the ties (see Figs. 45 to 52) give information on the distribution of bending moments. Since the curvature of the curve of flexure of the tie varies as the bending moment develops, the distribution of bending moment along the tie may be determined if the curvature at various points is known.

If R is the radius of curvature of the curve of flexure at a given point, in accordance with beam theory the expression for the curvature at that point is $\frac{1}{R}$ and the bending moment M will be $\frac{E\,I}{R}$, where E is the modulus of elasticity of the wood and I is the moment of inertia of the cross-section of the tie. It had been expected that the data of tie flexure would be of such nature as to give fairly accurate means of determining the moments developed in the ties and their distribution. It has been found that the local vertical compression of the wood at and near the rail, due to the bearing pressure of the rail, affects the depressions of the points near the rail (the amount was small, but the flexure of the tie was also small) to such an extent as to modify the form of the curve of flexure and thus throw some uncertainty on the radii of curvature derived therefrom, even at the middle of the tie. The values found for sections at the middle of the tie are fairly accurate; those at the rail are somewhat less reliable. The average of the values found will be useful for comparison. It seems probable that measurements made at the middle of the edges of the tie would give data of more value.

The curves of flexure of the various ties, obtained from the measurements, have been studied; the results give at least a qualitative distribution of the moments. To determine the modulus of elasticity of the wood, ten of the ties tested in the track were tested in the laboratory. The ties were considered to have a moisture condition similar to that existing when they were in the track. The tie was supported as a beam of 90-in. span, two equal loads applied at points 25 in. from the center, and the center deflection read over a chord of 40 in. The resulting values of the modulus of elasticity, calculated by the formula for deflection of beams, ranged from 1 000 000 to 1 400 000 lb. per sq. in., for the oak ties, averaging 1 250 000 lb. per sq. in. For the soft wood tie from Class B track, the value was 800 000 lb. per sq. in.

Table 17 gives values of bending moments in the tie at its center and at the rail, determined from the flexure curves by the method above described for the data of the ties tested in the track. The moments are expressed in pound-inches, W being the load carried by the tie, in pounds. As has been stated, it is to be expected that the method used is subject to error, and some of the values of the bending moment at the rail seem very high. The data are useful, however, in giving quantitative values which may be developed in track service.

It will be seen that negative bending moments as great as $-4.5\,W$ lb-in. were developed at the center of the tie, W being the load carried by one tie. For well tamped track and track kept in good condition, the negative moment will be considerably smaller, and for such conditions a value as low as $-2.0\,W$ may be expected as the upper limit. Excessive bearing pressures at the ends of the tie and needed strength

TABLE 17.—Values of Bending Moments in Tie Determined from Curves of Flexure.

W = Total load, in pounds, carried by one tie. Moment is then given in pound-inches.

Location.	Bending moment at center of tie.	Bending moment at rail.
Illinois Central Railroad		
Location *B*.		
Maximum value	$-3.7\ W$	$+4.0\ W$
Minimum value	$-1.7\ W$	$+1.4\ W$
Average value before tamping	$2.5\ W$	$+1.9\ W$
Average value after slight tamping	$-1.9\ W$	$+3.2\ W$
Joint ties, Location *B*, average value	$-2.0\ W$	$+1.4\ W$
Location *C*.		
Maximum value	$-4.1\ W$	$+4.8\ W$
Minimum value	$-0.3\ W$	$+1.7\ W$
Average value before tamping	$-1.4\ W$	$+2.9\ W$
Average value after slight tamping	$-1.1\ W$	$+3.4\ W$
Location *A*, average value	$-2.5\ W$	$+2.0\ W$
Location *K*.		
Maximum value	$-2.5\ W$	$+1.6\ W$
Minimum value	$-2.3\ W$	$+1.0\ W$
Average value	$-2.4\ W$	$+1.2\ W$
Chicago, Milwaukee and St. Paul Railroad		
Class A track.		
Maximum value	$-4.5\ W$	$+6.0\ W$
Minimum value	$-0.8\ W$	$+3.2\ W$
Average value	$-2.5\ W$	$+5.0\ W$
Class B track		
Maximum value	$-2.1\ W$	$+3.7\ W$
Minimum value	$-0.9\ W$	$+2.3\ W$
Average value	$-1.5\ W$	$+3.3\ W$

in handling it make it seem desirable to provide also for a positive bending moment of $+\ 1.0\ W$ at the center.

For uniform bearing along the tie, the value of the bending moment at the rail would be about $+\ 2.0\ W$ lb-in. for ties 8 ft. 6 in. long. In Table 17 values as high as $+\ 4.8\ W$ and $+\ 6.0\ W$ are found. It would appear that bending moments even larger than these values may be expected in track considered to be in ordinarily good condition. For first-class track in excellent condition it seems probable that a bending moment $+\ 2.5\ W$ at the rail should not be exceeded. The high values of bending moment at the rail obtained from the flexure curves are not readily explained; it seems probable that the depression of the fibers of the wooden tie under and near the rail, as indicated in Fig. 58, results in a greater flexure of the tie than would otherwise occur. Even though the bending moment found by the process used is, by reason of this condition under the rail, greater than that which exists by virtue of the bearing pressures between the rail and the end of the tie, the reduction in resistance to flexure produced by excessive depression of the fibers under the rail may be as much or more than the

discrepancy in bending moment, and hence values of the bending moment such as those given previously are not too large for use under the conditions found with wooden ties.

The bearing pressure under the tie of course will vary in intensity along its length. A study of the data of the tie flexure tests indicates that the maximum bearing pressure per unit of length of tie, under static loading, will not ordinarily exceed twice the average bearing pressure over the length of the tie, and that for track in good condition the maximum will be less than one and one-half times the average. These values relate to bearing pressures per unit of length of tie; as will be shown in the next chapter of the report the variation in intensity of pressure from the edge to the middle point of the width of the tie is considerable and the maximum intensity at the middle of the width may be much greater than the average.

It may be noted that the heaviest load on a tie generally occurs under the closely spaced drivers of a heavy freight locomotive, and the static load on one tie may be found approximately by dividing the weight on a driving axle by the number of ties which come within a driver spacing.

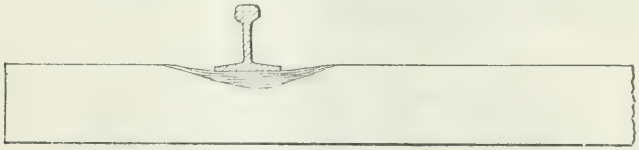


FIG. 58.—DIAGRAM SHOWING LOCAL DEPRESSION OF FIBERS UNDER RAIL.

It should be noted that the present discussion relates to static loading—no allowance has been made for the effect of speed, counterweight, and impact in increasing the loads on the tie; these should be considered in finding the bending moments, stresses, and bearing pressures which may develop in track service.

V.—TRANSMISSION OF PRESSURES IN BALLAST.

A.—Analytical Discussion.

27.—Introductory.—Crushed rock, gravel, cinders, burnt clay, slag, and sand are the materials commonly used for ballast. These materials are practically devoid of cohesion, at least when first placed. Earth and clay possess cohesive properties in some degree. Non-cohesive materials consist of separate pieces which will be hereafter referred to as grains. The grains are of irregular size and shape, and when first placed form a loose mass which cannot carry load without considerable readjustments of their relative positions. Under repeated applications of load, or other working, the grains are rearranged so that the mass becomes quite compact.

By reason of its structure, a load is transmitted through a non-cohesive granular mass from grain to grain by contact pressures. The

distribution of pressure is made through chains of grains in contact which lie between the load and a point in the ballast. Consideration will be given first to special forms of grains and then to the general form found in ballast.

28.—The Transmission of Load Through a Pile of Cylinders.— Assume that the cylinders are piled as compactly as possible. There are three cases to consider: First, when the cylinders are frictionless; second, when the coefficient of friction is equal to or greater than tan. 30°; third, when the coefficient of friction is between these two conditions.

In the first case, since there is no friction between the cylinders, the pressure at the point of contact must be normal to the surfaces in contact; that is, the line of action of the pressure must pass through the centers of the cylinders in contact. All the pressure, therefore, is transmitted through two chains, as shown in Fig. 59. It is evident that without adequate lateral support, the column of cylinders carrying the load will buckle, resulting in a rearrangement of the whole mass. The load-carrying capacity of a pile of frictionless cylinders is very small, because the only lateral forces which can be exerted are those due to the weights of the cylinders.

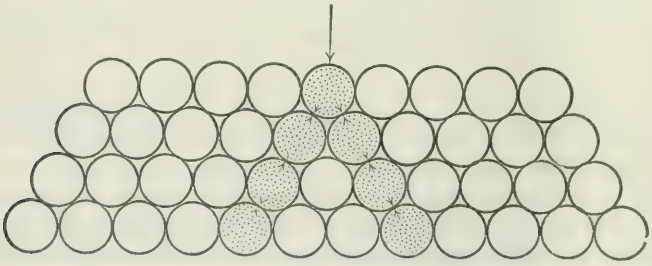


FIG. 59.—TRANSMISSION OF PRESSURES THROUGH PILE OF FRICTIONLESS CYLINDERS.

In the second case, when the coefficient of friction is equal to or greater than tan. 30°, all the load applied to the upper cylinder can be carried by the two cylinders below without the aid of lateral pressure. These in turn can be supported by one cylinder below them, as shown in Fig. 60 at (*a*). In this case there are no lateral forces developed or required for stability. That equilibrium will exist under this condition may be seen from the diagram at (*b*). The vertical force $\frac{1}{2}P$ may be resolved into two rectangular components. The ratio of the tangential to the normal component is

$$\frac{\frac{1}{2}P \text{ sin. } 30°}{\frac{1}{2}P \text{ cos. } 30°} = \text{tan. } 30°$$

For values of the coefficient of friction less than tan. 30°, slipping will occur; for equal or greater values, equilibrium will exist.

In the third case, when the coefficient of friction is greater than zero and less than tan. 30° the distribution of load will be between the extreme conditions represented by the cases previously considered. The actual distribution will depend upon the value of the coefficient of

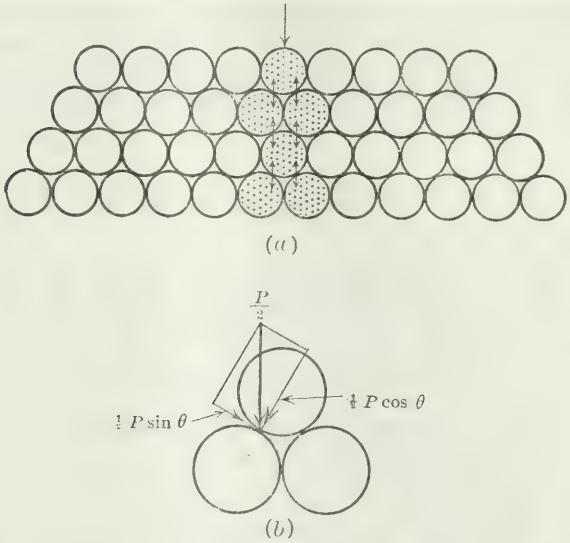


FIG. 60.—TRANSMISSION OF PRESSURE THROUGH PILE OF CYLINDERS WHEN COEFFICIENT OF FRICTION IS EQUAL TO OR GREATER THAN TAN. 30°.

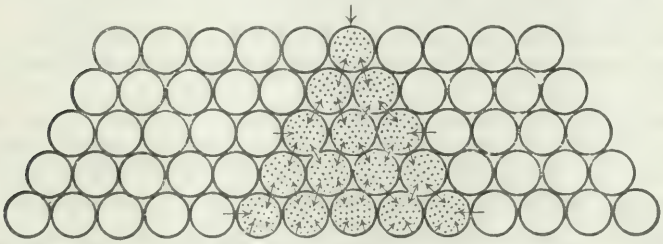


FIG. 61.—TRANSMISSION OF PRESSURE THROUGH PILE OF CYLINDERS WHEN COEFFICIENT OF FRICTION IS 0.2.

friction. As an illustration, the distribution of pressures in direction and magnitude when the coefficient of friction is 0.2 is indicated in Fig. 61. It will be noted that in this case lateral pressures are required in alternate rows in order to keep the mass in equilibrium.

It has been shown, then, that even when considering the distribution of pressure through a pile of equal-sized, incompressible cylinders, arranged symmetrically, the problem becomes so complex that it would be difficult to obtain results of general value. The complication is

caused by the effects of the lateral components of the pressures between the grains. This suggests that grains be assumed of such shape that there will be no lateral forces.

29.—The Transmission of Pressures Through a Pile of Grains in Which There are no Lateral Pressures.—To make the problem as simple as possible it will be assumed that the grains are of such shape that each grain rests on two grains and that the points of contact are always on the vertical center lines of the grains below, the distribution of the pressure being lateral and not longitudinal. Such a pile is shown in Fig. 62. One-half of the load applied to any grain is

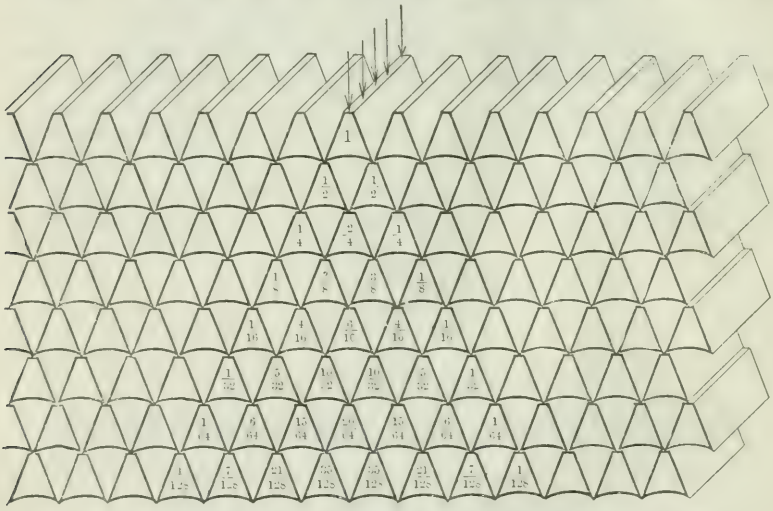


FIG. 62.—TRANSMISSION OF PRESSURE THROUGH PILE OF GRAINS WHEN NO LATERAL FORCES ARE DEVELOPED.

transmitted to each of the two grains below. In the first row below the loaded grain the pressure will be $\frac{1}{2}$ and $\frac{1}{2}$ of the applied load. In the second row below the loaded grain the pressures will be $\frac{1}{4}$, $\frac{2}{4}$, and $\frac{1}{4}$. In the third row it will be $\frac{1}{8}$, $\frac{3}{8}$, $\frac{3}{8}$, and $\frac{1}{8}$. In the nth row it will be:

$$\left(\frac{1}{2}\right)^n . n \left(\frac{1}{2}\right)^n . \frac{n(n-1)}{1 \times 2} \left(\frac{1}{2}\right)^n , \frac{n(n-1)(n-2)}{1 \times 2 \times 3} \left(\frac{1}{2}\right)^n , \text{ etc.}$$

That is, in any row, the pressures on the grains will be represented by the terms found by the expansion of the binomial

$$\left(\frac{1}{2} + \frac{1}{2}\right)^n \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

For example, in the bottom row of Fig. 62,

$$\left(\frac{1}{2} + \frac{1}{2}\right)^7 = \left(\frac{1}{2}\right)^7 + 7\left(\frac{1}{2}\right)^7 + \frac{7 \times 6}{1 \times 2}\left(\frac{1}{2}\right)^7 + \text{etc.}$$

It can be shown that the distribution of pressure given by this equation when n is large is similar to that given by the curve known as the probability curve. The equation giving this distribution of pressure on any horizontal plane is

$$p = \frac{k p_0}{\sqrt{\pi}} e^{-k^2 x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which p is the pressure at a horizontal distance x from the center line of the applied load, e is the Naperian base of logarithms, p_0 is the applied pressure and k is a function of the distance below the loaded grain. The form of the curves given by the equation may be judged from the three curves in Fig. 65.

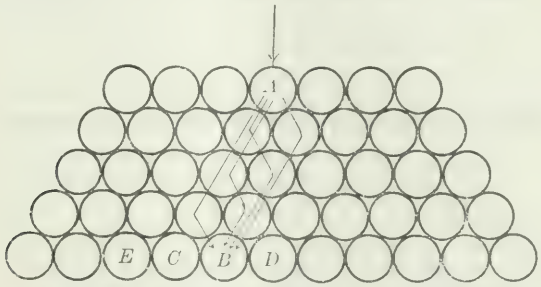


FIG. 63.—PATHS OF PRESSURE IN PILE OF CYLINDERS.

It is apparent that the condition assumed, each grain of ballast resting on the vertical center lines of two others, is a very special case. Results quite similar to those obtained above are found when the grains are of such form and arrangement that each grain rests on the vertical center line of three grains or of four grains. While the results found on such assumptions may not be quantitatively applicable, the analysis is of aid in forming a conception of the manner in which pressure is distributed and in giving a basis for expressing the results of experiments with ballast.

30.—Paths of Pressure and Law of Distribution of Pressure.—In general, there are a number of paths which the pressure may take as it is transmitted from one grain to another in the ballast; that is, there are a number of chains of grains in contact between the two grains under consideration. In Fig. 63 all of the possible paths of pressure between a loaded grain A and a grain B are indicated by lines. In this particular case there are four paths. Between Grains A and C there is only one path. Between A and E there is no path. Between A and D it can be shown that there are six paths. Referring also to Fig. 62 it will be seen that the number of chains of grains between A and B, C or D is the same as the numerator of the fractions representing

the pressures on the corresponding grains in the fourth row below the loaded grain.

In the preceding discussion, only grains of uniform size, special shape, and symmetrical arrangement have been considered. In ballast the grains have an infinite variety of sizes and shapes and are irregularly piled. Even with such irregular material the only method by which pressure can be transmitted from one grain to another not in contact with it is by means of chains of grains in contact with one another in the intervening space. Because of the irregular sizes, shapes and arrangements of the grains it is not possible to deduce a mathematical expression giving the amount of pressure transmitted from one grain to another in a particular case, but it has been found that by means of experiments and the use of the laws of averages good pre-

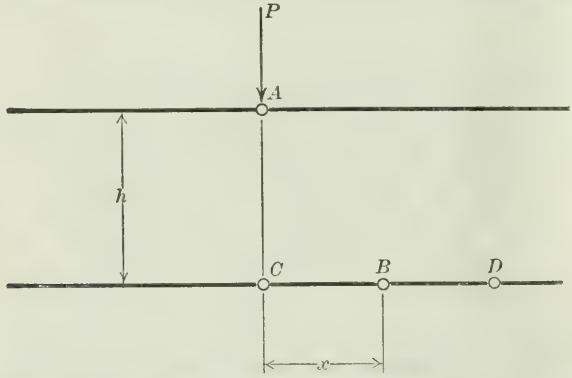


FIG. 64.—DIAGRAM SHOWING RELATIVE POSITIONS OF LOADED GRAIN AND POINTS ON HORIZONTAL PLANE.

dictions may be made. The following discussion of the probable distribution of pressure from a loaded grain has been found to accord with experimental results.

Assume a load P applied to a grain A at a height h above a horizontal plane on which there is a grain B, as shown in Fig. 64. Between A and B there are various paths of pressure, or chains of grains, through which pressure is transmitted. The number of paths, or chains, will in general increase as B is taken nearer to C (the grain directly under A) and will in general diminish as the distance x from C is increased until a point D is reached beyond which there is no chain of grains transmitting vertical pressure. Between A and C the number of paths is a maximum for that depth. In view of the discussion in the preceding articles, it seems reasonable to assume that the pressure transmitted to any grain will in general be proportional to the number of chains of grains between the two grains, and it may also be expected that in general the number of chains of grains from A to any grain B on the

given plane at any distance x from the center, and hence the pressure transmitted, will vary in accordance with the law of distribution of pressure represented by Equation (14) on page 1531. This law of the distribution of pressure is confirmed by the experiments.

This equation, then, may be taken to express in a general way the probable form of the distribution curve of the vertical pressure transmitted from a loaded grain in ballast consisting of grains of irregular size and shape and placed in any manner. It should be noted that the transmission is considered to take place only laterally, that is, in a vertical plane, it being virtually assumed that other similar transmission exists in adjacent parallel vertical planes, as would be the case with a cross-tie and ballast. In the case of tie and ballast, due to variation in bearing pressure along the length of the tie, there will be some longitudinal transmission of pressure, tending toward greater uniformity of pressure parallel to its length, but this action may be considered separately. For a load applied to a single grain, instead of to grains along a line, the transmission horizontally would be in every direction and the distribution would give equal intensities along the circumference of horizontal circles, making an entirely different distribution from the case under consideration.

Since Equation (14) is an expression of the pressure on any grain in terms of the pressure on a single grain and does not give intensity of pressure, it is not in form for use. Assume the load to be applied to the ballast by means of a long narrow strip one unit wide. The pressure on any grain in the ballast will then be the sum of the pressures which would be found by use of Equation (14) applied to each grain in contact with the strip, and the resulting intensity of pressure at any point in the ballast may be expressed by an equation of the same form. Let p_a be the applied unit pressure, let h be the depth of ballast above a horizontal plane, and let x be the horizontal distance from the line of action of the applied load to the point on the horizontal plane at which it is desired to determine the intensity of pressure p, then

$$p = \frac{k}{\sqrt{\pi}} p_a e^{-k^2x^2} \qquad (15)$$

k being a factor which depends upon h and must be found by experiment.

31.—Lines of Pressure.—In Fig. 65 curves have been constructed to represent the distribution of vertical pressure on several horizontal planes *ABC*, *DEF*, and *GHI*. The total areas under the curves represent the total upward reactions in the ballast on the horizontal planes. The upward reactions must be equal to the applied unit-load p_a when a section of ballast 1 in. long is considered. The areas under the several curves are therefore equal. These areas have been divided into a number of equal parts, each representing the same fractional part of the

reaction. The location of the resultant for each fractional area is noted on Fig. 65. Lines drawn through these points may be considered as lines of pressure representing the resultant fractional pressures.

The lines of pressure thus drawn are analogous to lines of force in a magnetic field, and the intensity of vertical pressure at any depth is seen to be equal to the number of lines of pressure passing through a width of 1 in. multiplied by the pressure which each line represents. For instance, if the lines are drawn representing pressures of 3 lb. and

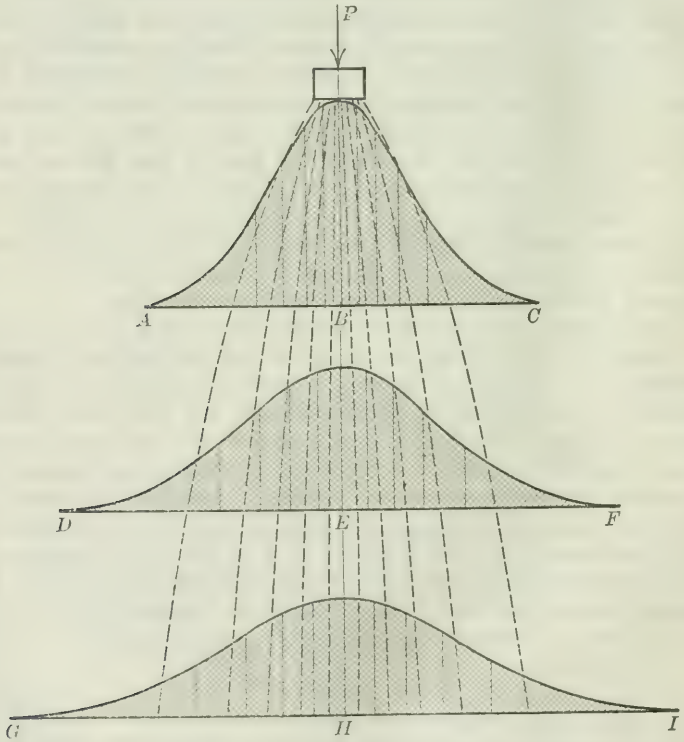


FIG. 65.—DIAGRAM ILLUSTRATING USE OF LINES OF PRESSURE.

at a given place on a horizontal plane four lines pass through a width of 1 in., the intensity of pressure is approximately $3 \times 4 = 12$ lb. per sq. in.

These lines of pressure will be found useful in explaining concentrations and distributions of pressure in ballast and in giving a conception of the manner of transmission of pressure.

32.—Rankine's Theory of the Relations of Pressures in a Granular Mass Devoid of Cohesion.—In 1856 Professor Rankine, from a study

of certain properties of stress relating to a infinitesimal wedge of granular material, enunciated several important theorems which have since borne his name. Several of these bear upon the discussion. They may be stated as follows:

Conjugate Stresses.*—*If any state of stress exists at a point in a granular mass, the stress acting on a plane through the point will be parallel to a second plane through the point, and the stress acting on the second plane will be parallel to the first plane. Stresses so related are called conjugate stresses.*

In Fig. 66, let the elementary prism be in equilibrium when acted upon by the stresses p parallel to AC, and the stresses q, direction unknown. It is required to find the direction of stresses q. Now the stresses p acting on the faces AB and CD must constitute a pair of forces in equilibrium; then to have equilibrium stresses q acting on the faces AC and BD must form a pair of forces in equilibrium, which can only occur when the stresses q are parallel to AB and CD.

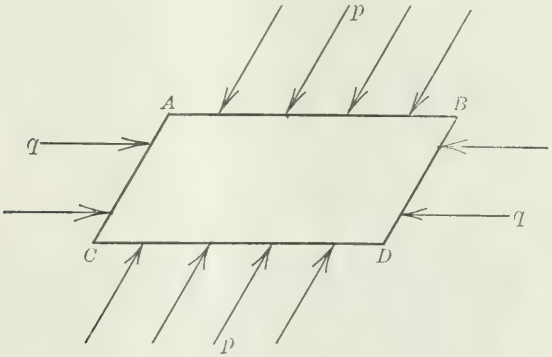


FIG. 66.—PARALLELOPIPED UNDER ACTION OF CONJUGATE PRESSURES.

Principal Stresses.*—*In any state of stress there is one pair of conjugate stresses at right angles to each other; i. e., there are two planes, at right angles to each other, on which the stress is normal only. Stresses so related are principal stresses.* For consider a plane at the point to rotate. There will be some position where the stress on the plane is normal; the plane at right angles will have a conjugate stress which is also normal. It can be shown that one of the principal stresses is the greatest stress and the other principal stress is the least stress at the point. Any possible state of stress can be completely defined by principal stresses.

* Taken with slight modifications from Ketchum's "Walls, Bins and Grain Elevators", to which the reader is referred.

The Ratio of Conjugate Pressures.—*The ratio of the intensity of the pressures on any pair of conjugate planes of pressure when motion is impending, is expressed by the equation:*

$$\frac{p_b}{p_a} = \frac{\cos.\theta - \sqrt{\cos.^2\theta - \cos.^2\phi}}{\cos.\theta + \sqrt{\cos.^2\theta - \cos.^2\phi}} \quad \ldots\ldots\ldots\ldots (16)$$

in which the stress p_a is greater than its conjugate unit stress p_b, θ is the common angle of obliquity which the stresses make with the normals to their planes, and ϕ is the angle of repose of the material.

When the angle θ is taken as zero, the conjugate stresses are perpendicular to their planes, hence p_a and p_b are the principal stresses. When motion is impending,

$$\frac{q}{p} = \frac{1 - \sin.\phi}{1 + \sin.\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which p and q are the greatest and least principal stresses, respectively.

Equations (16) and (17) express the relationships existing between conjugate pressures and principal pressures when motion is impending. The principal pressure q and the pressure p_b are *active pressures* developed by the application of their conjugate pressures. In case the resisting pressure is less than the active pressures the granular material will flow. In case the lateral pressures are greater than indicated by the equations, the granular mass will be in equilibrium.

33.—Conditions Accompanying the Application of Pressure by Means of a Tie.—In the preceding discussion it has been assumed that the pressure is applied only to a narrow strip. When the pressure is applied to the ballast by means of a tie the load is applied of course to the many grains in contact with the tie. It is common to consider that the intensity of pressure is the same all over the bottom of the tie and that it is normal to it (vertical). There is, however, a variation of the intensity of the pressure on the bottom of the tie due to the tendency of the ballast to move from the center to the edge of the tie when load is applied. Friction between tie and ballast develops pressures on the ballast which are not vertical. The maximum variation of pressure due to these causes will occur when the load is so great that the tie is being forced into the ballast. The following analysis will indicate the law of distribution under this condition of loading.

34.—Variation of Intensity of Pressure on the Bottom of the Tie When the Tie is Being Forced into the Ballast.—The friction between the tie and the ballast is probably less than the friction between the particles of the ballast, hence the ballast in contact with the bottom of the tie will be the first to move when the tie is heavily loaded. Consider a section of tie one unit long. Let the vertical unit-pressure at the edge be p_o, at the center be p_c, and at a distance x from the edge

be p. Let the coefficient of friction between the tie and the ballast be f, and the width of the tie be b. Let the total load on the tie be P and let l be the length of the tie. When motion is impending the intensity of pressure increases from the edge to the center of the tie because the intensity of vertical pressure at any point is limited by the available lateral pressure.

In Fig. 67 at (a), p_a is the resultant intensity of pressure at any point on the bottom of the tie when motion is impending. Expressed in terms of the vertical unit-pressure p at the point, the value of p_a is

$$p_a = \frac{p}{\cos.\ \theta}$$

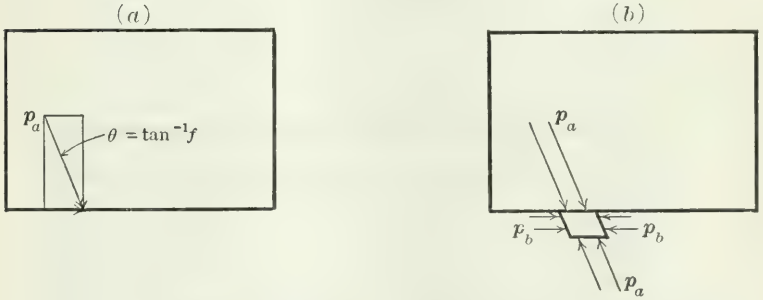


FIG. 67.—DIAGRAMS ILLUSTRATING PRESSURES ON BOTTOM OF TIE.

The horizontal pressure in the ballast at the point is the conjugate of the pressure p_a and may be found from Equation (16). For given values of θ and ϕ, using k as the resulting constant,

$$p_b = k p_a$$

in which

$$k = \frac{\cos.\ \theta - \sqrt{\cos.^2 \theta - \cos.^2 \phi}}{\cos.\ \theta + \sqrt{\cos.^2 \theta - \cos.^2 \phi}}$$

The total frictional force from the edge of the tie to a point x distant is equal to the horizontal thrust $\frac{k p}{\cos.\ \theta}$. In the distance dx, Fig. 68, the added friction amounts to $f\ p\ dx$, which can hold in equilibrium an additional horizontal thrust equal to

$$\frac{k^2 dp}{\cos.\ \theta} = f\ p\ dx$$

Hence

$$\frac{k}{\cos.\,\theta}\int_{p_0}^{p}\frac{dp}{p} = f\int_0^x dx$$

Integrating,

$$\frac{k}{\cos.\,\theta}\log._e\frac{p}{p_0} = f\,x$$

$$\frac{p}{p_0} = e^{\frac{f\,x\,\cos.\,\theta}{k}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

where e is the Naperian base of logarithms. The vertical unit-pressure at the center is

$$p_c = p_0\,e^{\frac{f\,b\,\cos.\,\theta}{2\,k}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

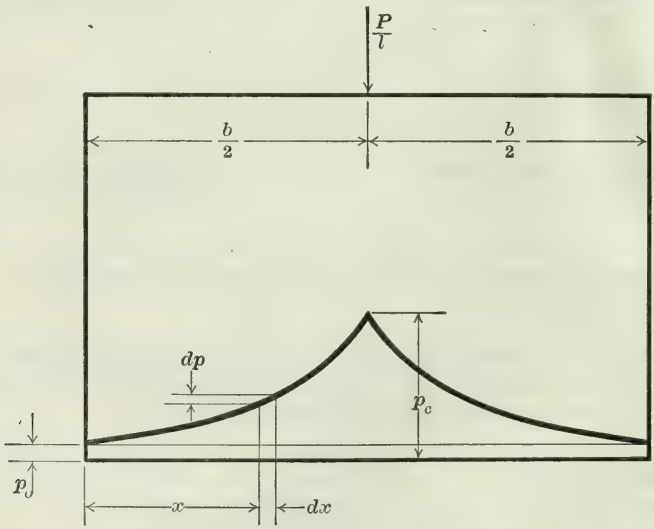


FIG. 68.—DISTRIBUTION OF VERTICAL UNIT-PRESSURE ON TIE WHEN CARRYING ULTIMATE LOAD.

The total load which can be carried by the section of the tie without its being forced into the ballast is equal to the area under the curve in Fig. 68.

$$\frac{P}{l} = \frac{2\,p_0\,k}{f\,\cos.\,\theta}\left(e^{\frac{f\,b\,\cos.\,\theta}{2\,k}} - 1\right) \qquad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

This equation gives the ultimate load, P, which the tie will support.

The average vertical unit-pressure carried by the tie is

$$\frac{1}{b}\cdot\frac{2\,p_0\,k}{f\,\cos.\,\theta}\left(e^{\frac{f\,b\,\cos.\,\theta}{2\,k}} - 1\right) \qquad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The ratio of the vertical unit-pressure at the center to the average vertical unit-pressure on the tie is

$$\frac{\frac{p_c}{P}}{\frac{}{b\,l}} = \frac{f\,b\cos.\,\theta\; e^{\frac{f\,b\cos.\,\theta}{2\,k}}}{2\,k\left(e^{\frac{f\,b\cos.\,\theta}{2\,k}} - 1\right)} \qquad (22)$$

In making numerical calculations it is best to change from the Naperian base to the base of common logarithms by means of the equation

$$e^{\frac{f\,b\cos.\,\theta}{2\,k}} = (10)^{\frac{f\,b\cos.\,\theta}{4.6\,k}}.$$

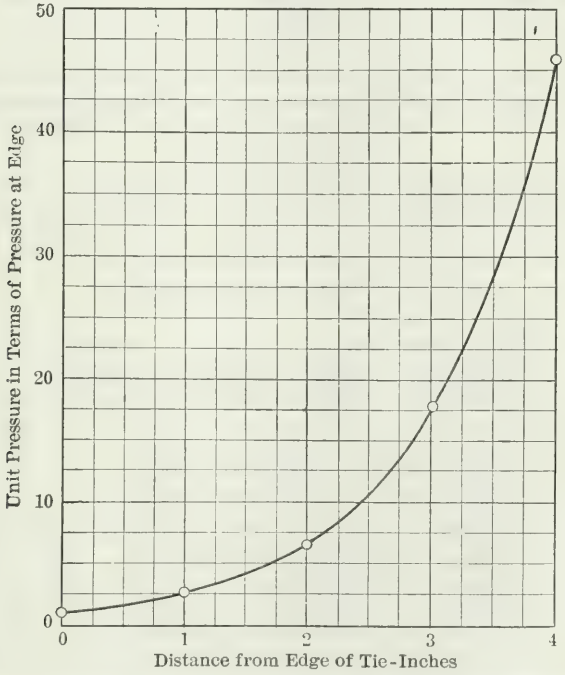


FIG. 69.—DIAGRAM SHOWING RELATIVE PRESSURES ON TIE WHEN CARRYING ULTIMATE LOAD FOR TAN. $\phi = 2/3$ AND TAN. $\theta = 0.4$

Values calculated from Equation (18) have been plotted in Fig. 69, which gives the greatest intensity of pressure that can exist at points away from the edge of the tie in terms of the intensity of pressure at the edge of the tie for assumed conditions of friction. It illustrates the rapid increase in intensity from the edge of the tie inward, which could be developed under the assumed conditions, when the tie is on the point of being forced into the ballast.

The ultimate supporting power of the tie is shown by Equation (20) to increase rapidly with the width. Using the assumptions here made, Table 18 gives the relative ultimate supporting capacity of ties of various widths, for tie on point of being forced into the ballast, based on an angle of repose of $1\frac{1}{2}$ to 1 and on a coefficient of friction between tie and ballast of 0.4. This is intended to represent approximately the conditions during quiescent loading. The relative ultimate supporting capacity of ties of various widths, for tie on point of being forced into the ballast, based on an angle of repose of 3 to 1 and on a coefficient of friction between tie and ballast of 0.2, is also given. This is thought to approximate the conditions when the application of load is accompanied by jarring as in the case of train loads. The exact values of the friction angles are not known. Table 18, of course, has an analytical, rather than an experimental basis.

TABLE 18.—Relative Supporting Capacities of Ties of Different Widths, for Quiescent Loadings and for Loadings Accompanied by Jarring, for Tie on Point of being Forced into the Ballast.

Quiescent load condition is based on assumed coefficient of 0.4 between tie and ballast and of $\frac{2}{3}$ between particles of the ballast. For jarring loads values of 0.2 and $\frac{1}{3}$ are assumed.

Width of tie, inches.	Ultimate Supporting Capacity.			
	In terms of p_0.		In terms of 8-in. tie.	
	Quiescent.	With jarring.	Quiescent.	With jarring.
1	1.27	1.08	0.01	0.07
2	3.33	2.34	0.04	0.15
3	6.65	3.79	0.07	0.24
4	12.1	5.55	0.13	0.35
5	20.7	7.55	0.22	0.47
6	34.8	10.0	0.37	0.62
7	57.4	12.7	0.61	0.80
8	93.7	16.0	1.00	1.00
9	153	19.1	1.63	1.20
10	248	24.2	2.65	1.51
11	401	29.5	4.27	1.85
12	650	35.2	8.94	2.20

Except when heavy loads are carried on loose ballast, the distribution of pressure on a railroad tie is not in general that shown in Fig. 69, because the tie is not at the point of being forced into the ballast. After the track has been in service for a time the ballast beneath the tie will become compacted. The carrying capacity of the ballast at the edge of the tie will then have been increased, and since the ultimate carrying capacity of the tie varies directly with the carrying capacity of the ballast at the edge of the tie, the ultimate carrying

capacity will also have been increased over what it could carry when the ballast was first placed.

At (*a*) in Fig. 70 is shown the distribution of pressure on the bottom of the tie when it is carrying its ultimate load. Vectors giving the directions and relative magnitudes of the unit-pressures are shown. The ordinates of the full line curve represent the vertical components of these pressures. When the tie is on the point of being forced into the ballast, the angle which the pressure makes with the normal to the bottom of the tie is the angle of friction between tie and ballast. The ultimate vertical unit-pressure which may be developed on the bottom of the tie after the ballast has become compacted is indicated by the dotted curve at (*b*) in Fig. 70. The distribution and direction of pressure on the tie under ordinary loadings (that is, loadings less than the ultimate load which the ballast can carry) will be somewhat as shown by the full line curve and the vectors. It is seen that the distribution shown is different from that at ultimate load.

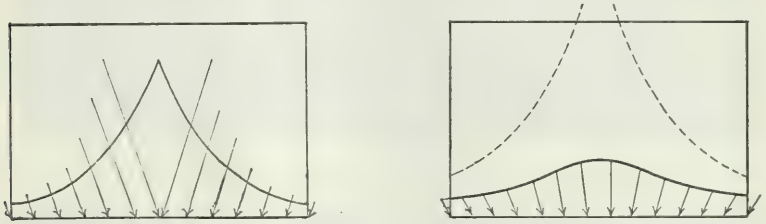


FIG. 70.—DIAGRAM SHOWING RELATIVE DIRECTIONS AND MAGNITUDES OF PRESSURES ON TIE UNDER ULTIMATE LOAD AND UNDER ORDINARY LOADS.

The coefficient of friction between the tie and ballast and between parts of the ballast will be much less under the jarring and pounding action of train loads than under loads applied without shock or vibration. The ballast gradually works from under the tie and in time the track must be retamped. From the discussion it is evident that in tamping it is not enough to force ballast beneath the tie unless the ballast at the edge of the tie is compacted, otherwise the track will settle rapidly. Especial attention therefore should be given to methods of tamping which insure a compacted ballast at the edge of the tie.

35.—Method for Finding the Distribution of Pressure in Ballast Beneath a Cross-Tie.—It has been shown that the pressure in ballast against the tie is greater near the center than at the edge of the tie and that because of the friction between tie and ballast the pressures are not perpendicular to the bottom of the tie. It has also been shown that the distribution of pressure in granular material when a narrow strip is loaded may be represented by an expression of the type of the probability equation, or by lines of pressure. By dividing the bottom surface

of the tie into narrow strips and using the average load on each strip and taking its direction into account, the total intensity of pressure at any point in the ballast may be found by adding the pressures at that point due to the loads on the several strips.

In order to use Equation (15) to find the vertical pressure at any point B in the ballast due to a pressure p_a at A, Fig. 71, when p_a is not vertical, it is necessary to determine the distances x_1 and h_1, and use them for x and h in Equation (15). It may be shown from the geometrical relations that

$$x_1 = (h \tan. \theta - a + x) \cos. \theta \quad \ldots\ldots\ldots\ldots\ldots (23)$$

$$h_1 = \frac{h - x_1 \sin. \theta}{\cos. \theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

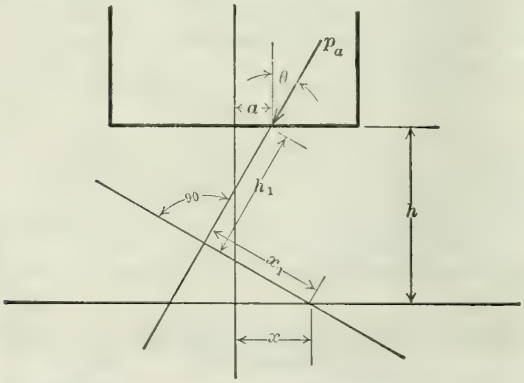


FIG. 71.—DIAGRAM FOR DETERMINING x_1 AND h_1.

The pressure p found from Equation (15) is in the direction of the pressure p_a. The vertical component of the pressure is found by multiplying the pressure found by cos. θ.

The pressure p_a at any point on the bottom of the tie probably should be taken as the greatest principal pressure at the point. The greatest principal pressure will make a greater angle with the vertical than does the conjugate pressure and its magnitude is greater. However, due to the uncertainties in the magnitude and direction of the pressures on a tie, it is a needless refinement to take account of the principal pressures, and values of the conjugate pressures will be used instead.

B.—Experimental Work in Laboratory.

36.—Earlier Experiments at the University of Illinois.—Experimental work on the transmission of vertical pressure through granular material was carried on for a number of years at the University of

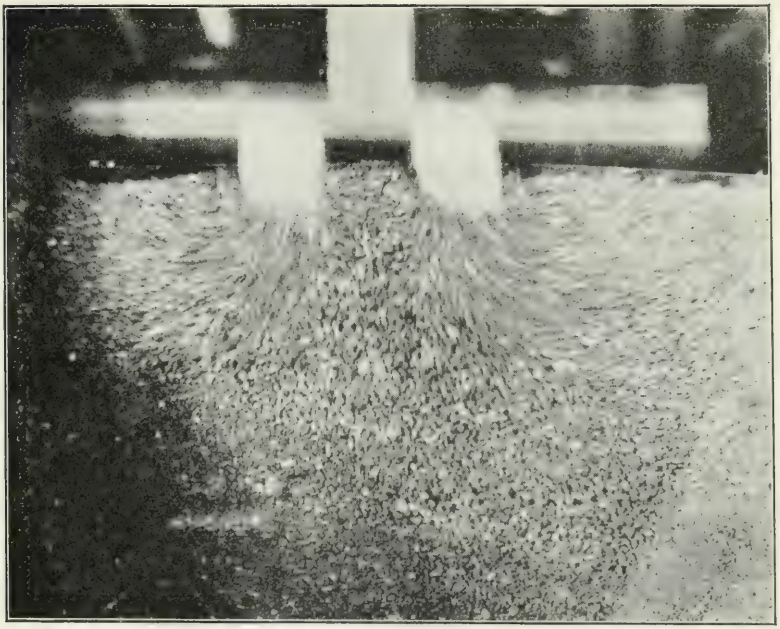


FIG. 72.—LINES OF MOVEMENT OF SAND BALLAST UNDER LOAD.

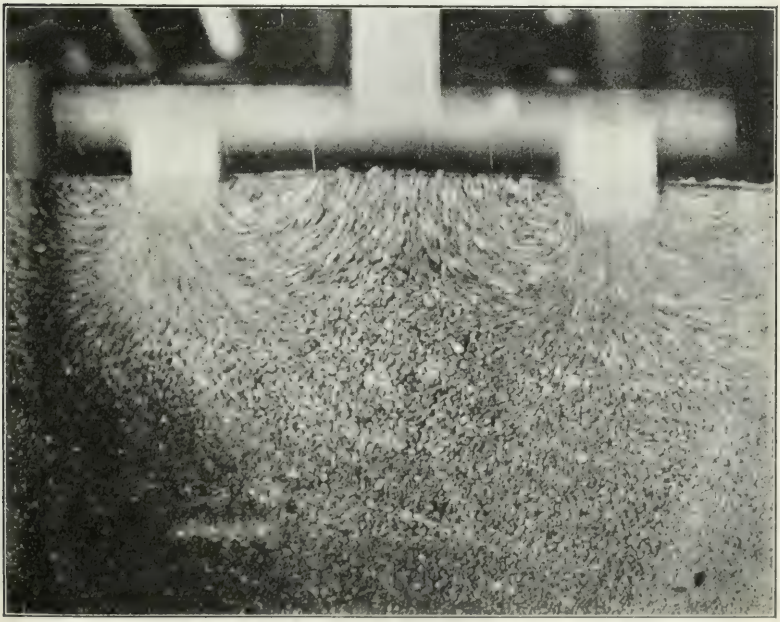


FIG. 73.—LINES OF MOVEMENT OF SAND BALLAST UNDER LOAD.

Illinois in connection with undergraduate thesis work.* Although the experiments were limited to circular bearing areas and therefore are not directly applicable to railroad track, the work proved very instructive and the experience gained was helpful in planning the more elaborate experimental work afterward undertaken for the Joint Committee.

Figs. 72 and 73 are from photographs taken by Messrs. L. N. Fisher and H. F. Wagner in 1912 and included in their thesis. The photographs were made by taking a time exposure of coarse sand behind a plate of glass while a load was being applied to two small blocks (2 by 2 in.). The images of the moving grains are elongated in the direction of motion, giving the appearance of lines of force in a magnetic field. The photographs indicate the paths of flow in the case of the very loose ballast used and, in a general way, indicate the tendencies of ballast movements in track under the action of load.

37.—The Laboratory Experimental Work.—The laboratory experimental work undertaken by the Committee was made in the Laboratory of Applied Mechanics of the University of Illinois. In the attempts which had been made to measure pressures in ballast under regular track conditions, difficulties were met that showed the advisability of conducting experiments in the laboratory where conditions could be controlled and results obtained to establish some of the basic principles of the action of ballast.

Two elevations showing the arrangement of the apparatus used in making the tests are shown in Fig. 74. A reinforced concrete slab 16 by 19 ft. and 12 in. deep supported the ballast. The load was applied to the ties by means of a jack whose upward thrust was carried by I-beams to four $1\frac{7}{8}$-in. steel rods having ends embedded in the concrete slab. The load applied by the jack was distributed to the one, two, or three ties used in the experiments by means of a system of levers. It was possible to divide the load among the ties in any manner desired. The load applied by the jack was measured by a calibrated steel spring. The arrangement of the apparatus can be seen in Fig. 75.

For measuring the intensity of pressure transmitted to various parts of the ballast, the pressure capsule shown in Fig. 76 was used. The following description which was given in the first progress report is here repeated for the convenience of the reader:

The elastic deflection of a thin steel diaphragm is used to measure the amount of pressure applied to the capsule. The pressure to be measured is received on the bearing plate *P*, which has an area of 5 sq. in.; the pressure is transmitted to the thin steel diaphragm *D*, which is fastened by screws around its circumference to the cast-

* An article by Prof. M. L. Enger describing the methods of experimenting and giving the results was published in the following papers: *Engineering Record*, Vol. 73, p. 106, January 22d, 1916; *Railway Age Gazette*, Vol. 60, p. 321, February 18th, 1916; *Engineering and Contracting*, Vol. XLV, p. 53, January 19th, 1916; and *Railway Review*, Vol. 58, p. 129, January 22d, 1916.

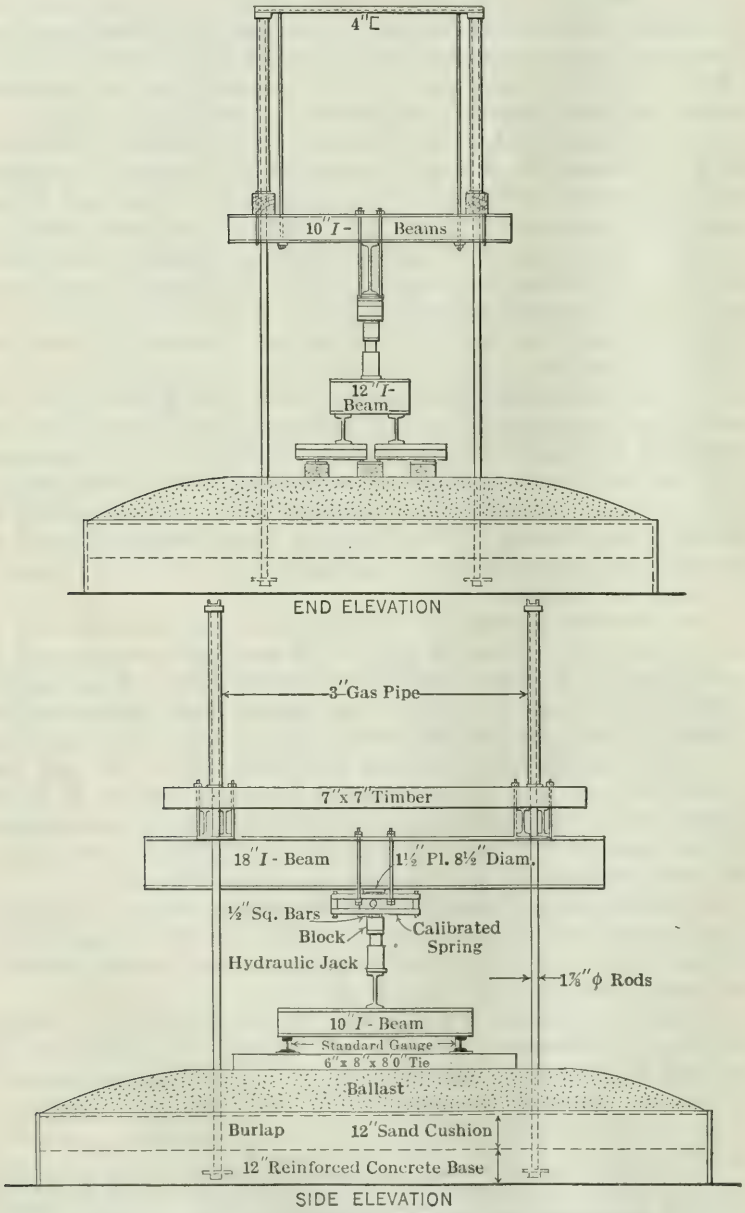


FIG. 74.—APPARATUS USED IN TESTS ON TRANSMISSION OF PRESSURE THROUGH BALLAST.

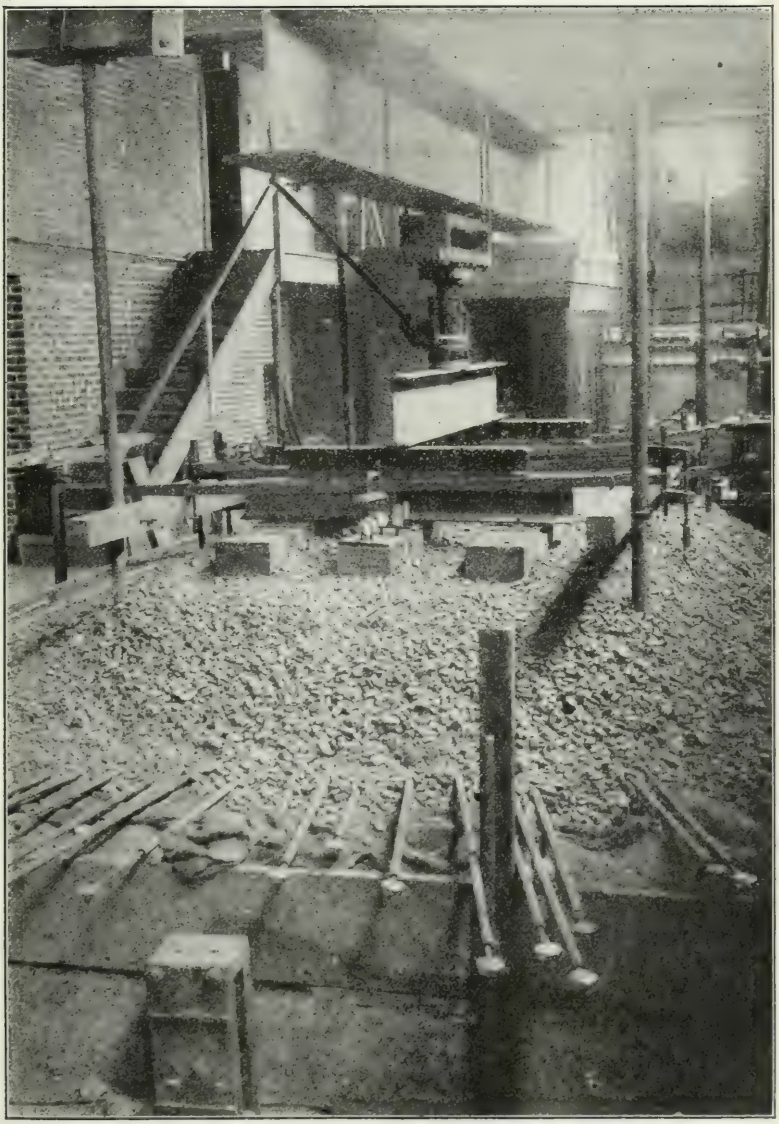


FIG. 75.—APPARATUS USED IN TESTS ON TRANSMISSION OF PRESSURE THROUGH BALLAST.

iron case B. The screw S which fastens the plate P to the diaphragm D is hardened and bears on one knife edge of the small bell-crank lever L which is pivoted at Q. The vertical deflection of the center of the diaphragm is transmitted by the bell-crank lever (magnified about three times) in a horizontal direction to the rod R which slides in guides NN' and is enclosed in a horizontal tube K and finally bears against plunger T of an indicating dial micrometer, whose movement

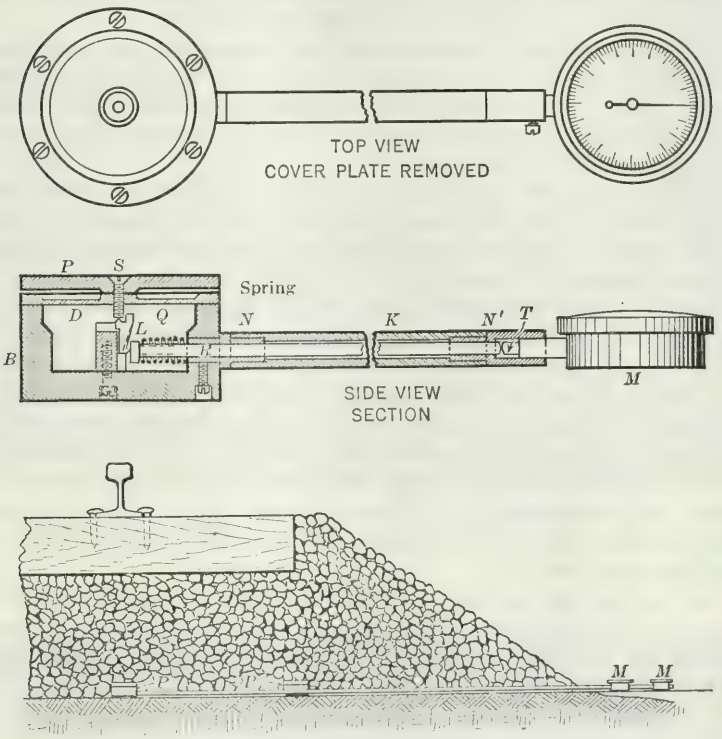


FIG. 76.—PRESSURE CAPSULE AND ITS PLACE IN BALLAST.

is thus a measure of the elastic deflection of the diaphragm D. If the material of the diaphragm is not stressed beyond its elastic limit the deflection of the diaphragm, and the consequent movement of the pointer of the dial micrometer, may be used to measure the load on the plate P. Sand and dirt are prevented from coming between the bearing plate P and the diaphragm by friction tape wrapped around the case of the pressure capsule. Each pressure capsule was calibrated after being taped, by placing it on a platform scale and by means of a screw clamp applying a series of loads covering the range of its use.

The thin elastic diaphragm is made of hardened steel. The bearing plate serves to take load of any distribution or concentration upon it and to apply this load as a concentrated load at the center of the diaphragm.

There was at first some difficulty due to ballast movements causing the tube to be pulled out of the capsule. Even after making a threaded connection between the tube and capsule there was some trouble from this cause. It was finally entirely overcome by slipping a 1-in. pipe over the tube, protecting it from the pull of the ballast.

38.—Procedure of Tests.—During the progress of the work some changes were made in the making of the tests. The most important changes relate to the disposition of the pressure capsules in the ballast. The locations of the capsules are shown in each case in the diagrams giving the results of the tests.

In the experiments with the sand ballast, the capsules were placed on the concrete slab and great care was taken in placing the sand around them in order to prevent them from being displaced. After a sufficient quantity of ballast had been placed it was struck off to a level surface at the required depth. After the ties had been placed carefully in the desired position with respect to the capsules, they were loaded five or six times before beginning to take readings. The micrometer dials were then read with no load on the ties. A small load was then applied and readings were again taken. Usually a zero load and three increasing loads were applied. Readings were taken until three consecutive sets of consistent readings were obtained. The greatest load to be applied was determined by: (*a*) the capacity of one or more of the pressure capsules, (*b*) the carrying capacity of the ballast, or (*c*) the capacity of the jack.

In the first test the sand was placed so that it was 6 in. from the bottom of the tie to the top of the capsule. When the experiments with this depth of ballast had been completed the pressure capsules were rearranged and experiments were made with depths of ballast of 9, 18, and 25 in. The ballast was loosened up in each case before additional sand was put on.

Following these tests, the ballast was removed. The capsules were recalibrated and given a different arrangement (see Fig. 77) and were covered with sand ballast to a depth of 12 in. from bottom of tie to top of capsule.

Depths of broken stone ballast of 12 in. and 18 in. were next used. In these experiments a 15-in. sand cushion was placed on the concrete slab and covered with burlap. The capsules were placed on the burlap as shown in Fig. 78. Then the broken stone was carefully placed so as not to disturb the capsules. The experiments were made in the same manner as with sand ballast.

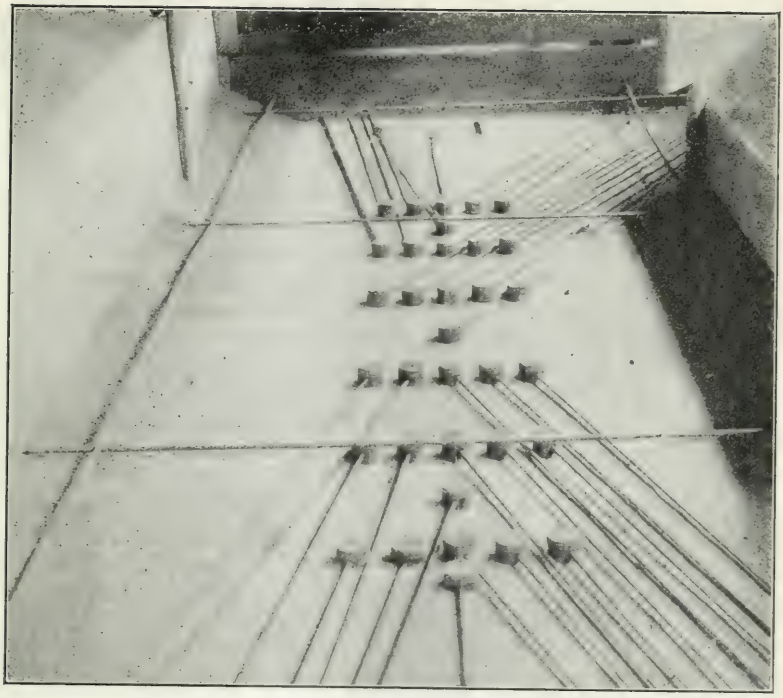


FIG. 77.—ARRANGEMENT OF CAPSULES AND RODS UNDER SAND BALLAST.

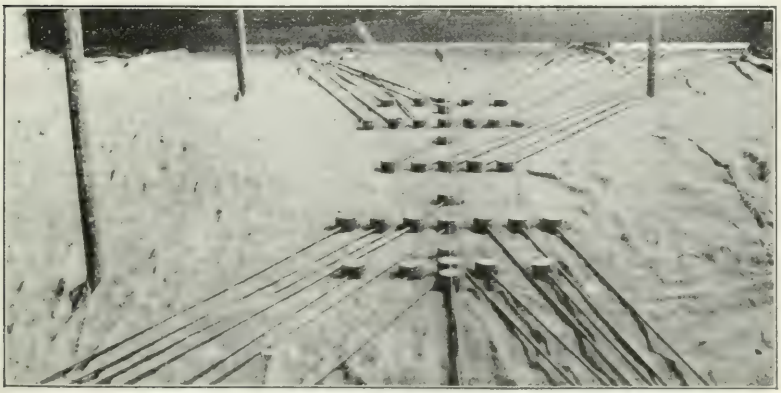


FIG. 78.—ARRANGEMENT OF CAPSULES FOR TESTS WITH BROKEN STONE BALLAST.

Pebble ballast having a depth of 12 in. from tie to capsule was next used. The capsules were recalibrated and were placed on the burlap with a slightly different arrangement than was used in the broken stone ballast.

39.—Results of Tests.—The results of the tests are given in Figs. 79 to 87, presented graphically in order to facilitate comparison and to give the essential information in the most compact form. For example, in Fig. 80, there are given the results of the tests on the transmission of pressure from a single tie through 12 in. of sand ballast. The relative position of the tie and the capsule is shown, and in addition there are curves giving the vertical pressure at various sections. The curves at *AB* represent the averages of the pressures measured by the capsules at *AB* and *LM*. To illustrate, the pressures determined from the readings of the Pressure Capsules 1, 2, 3, and 4, were averaged and are plotted above Capsules 1 and 2. The curves above *CD* represent the average pressures as measured by the capsules on the lines *CD* and *JK*, and the curves above *GH* represent the average pressure measured on *EF* and *GH*. Below the plan view of the tie are plotted curves representing the pressures 12 in. below the center line of the tie. By the manner of plotting all the curves are symmetrical. The dotted lines in Fig. 80 represent the intensity of pressure when the tie carries a total load of 35 000 lb. The full lines are for a total load of 24 000 lb.

Fig. 79 gives the results of experiments on a single tie with depths of sand ballast of 6, 9, and 18 in., Fig. 80 gives results for sand ballast 12 in. deep when the ordinary 6 by 8-in. by 8-ft. tie is used and also when a tie with a wedge-shaped bottom is used. The wedge was $1\frac{1}{2}$ in. high in the middle of the tie. It will be noted that the pressure is somewhat more widely distributed with the wedge-shaped tie.

Figs. 81, 82, and 83 give the results of the experiments for depths of sand ballast of 9, 18, and 25 in. when the ties are equally loaded and when the middle tie carries twice the load carried by either of the outside ties.

Fig. 84 gives the results of the experiments for 12 in. of broken stone ballast and for the same depth of pebble ballast. The broken stone used was the standard limestone ballast material furnished by the Illinois Central Railroad. The pebble ballast consisted of pebbles ranging in size from about $\frac{1}{4}$ in. to $1\frac{1}{2}$ in. and was free from sand. It will be noted that the results of experiments with rock ballast are more erratic than with pebbles or with sand. This was to be expected because a short chain of the large grains may bring a heavy pressure on a capsule, or it may miss it entirely. When average pressures are taken there is a surprising similarity in the distribution of pressure through sand, pebble, and broken stone ballast.

Fig. 85 shows the distribution of pressure through 12 in. of broken stone when the load is carried on a 6 by 10-in. by 8-ft. tie with the 10-

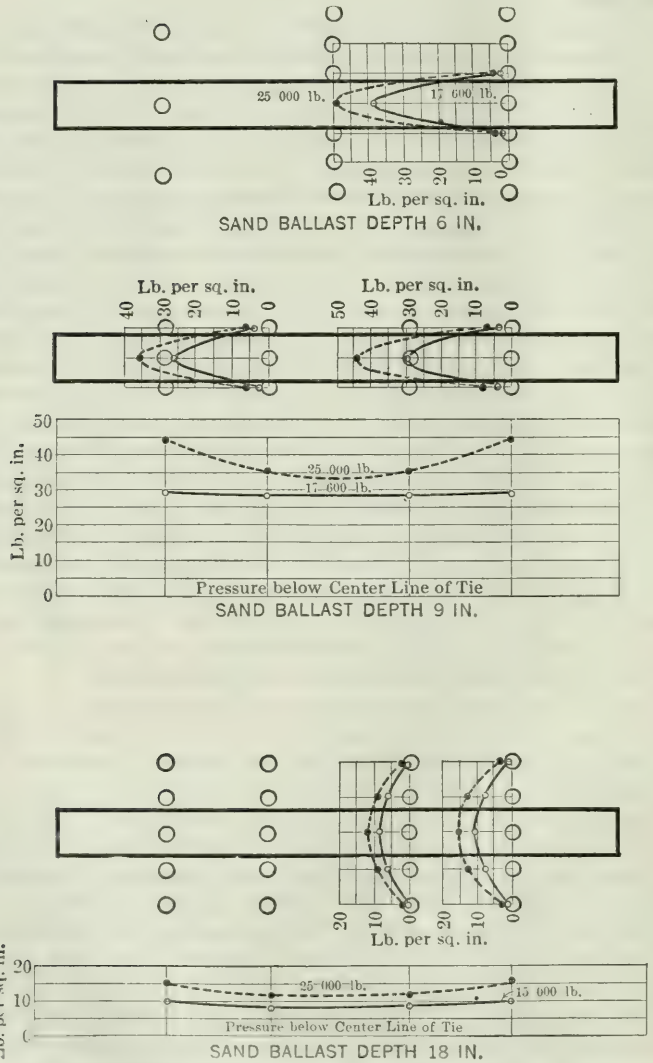
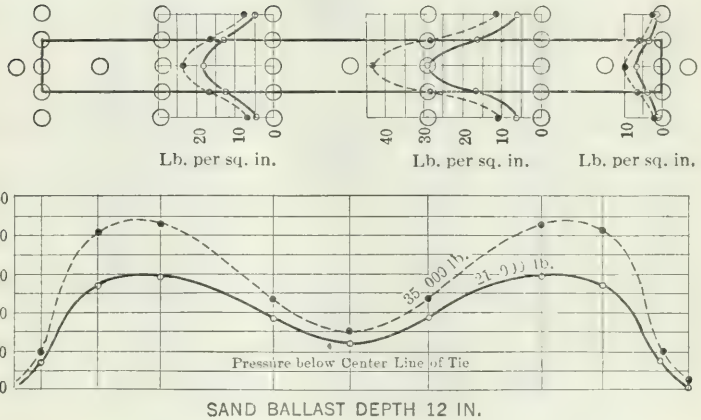


FIG. 79.—DISTRIBUTION OF PRESSURE AT DEPTHS OF 6, 9, AND 18 IN. OF SAND BALLAST BELOW A SINGLE TIE.



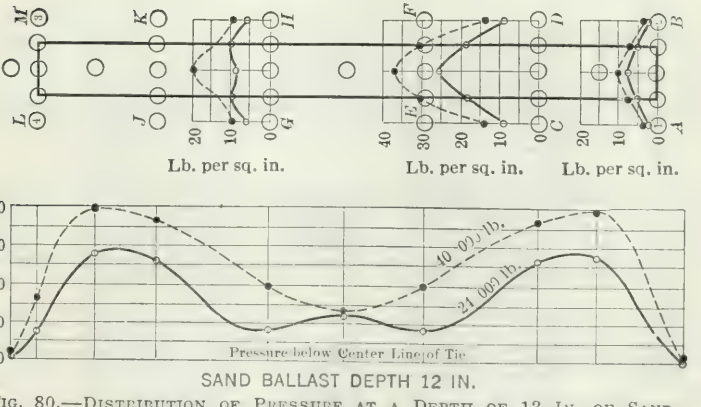
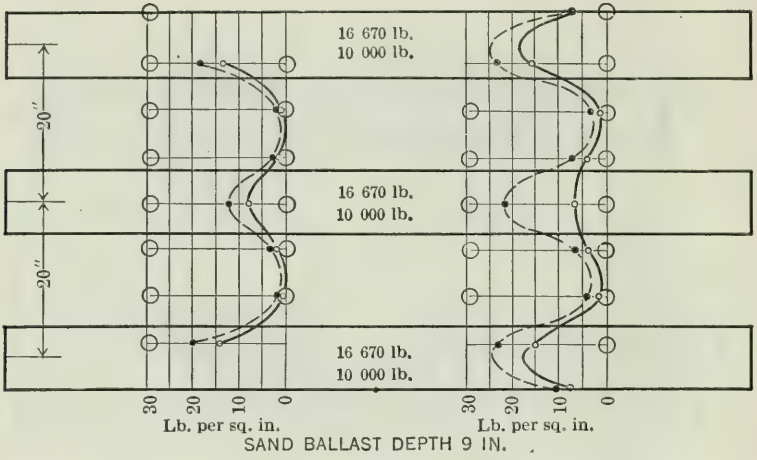


FIG. 80.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 12 IN. OF SAND BALLAST FOR A SINGLE ORDINARY TIE AND A SINGLE TIE WITH WEDGE-SHAPED BOTTOM.



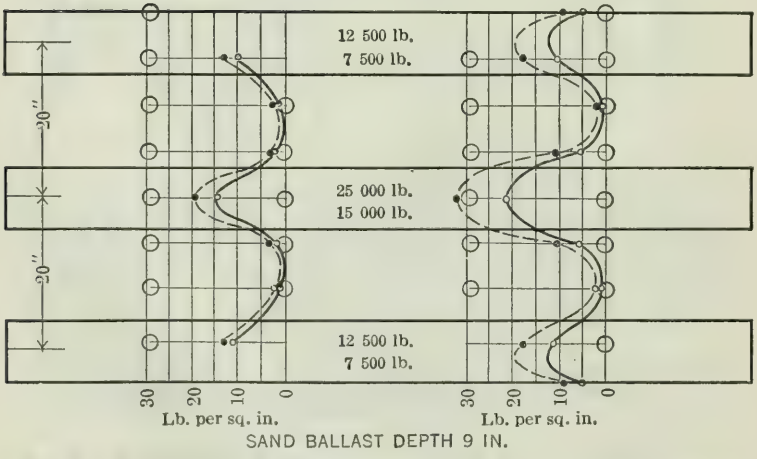
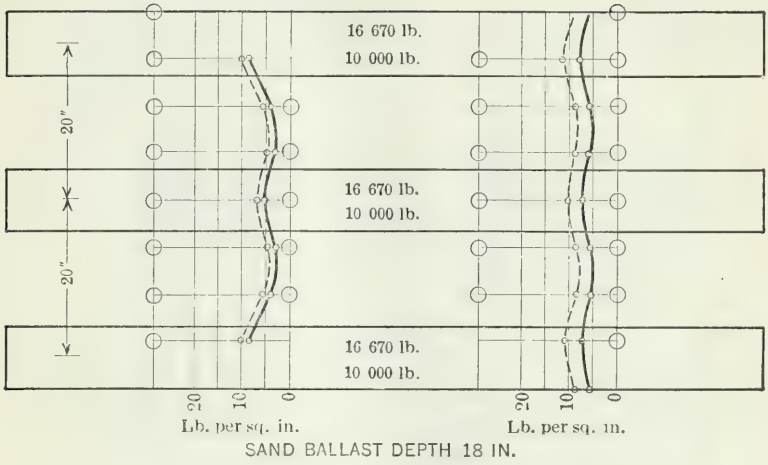


FIG. 81.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 9 IN. OF SAND BALLAST BELOW THREE TIES. EQUAL AND UNEQUAL LOADS ON TIES.



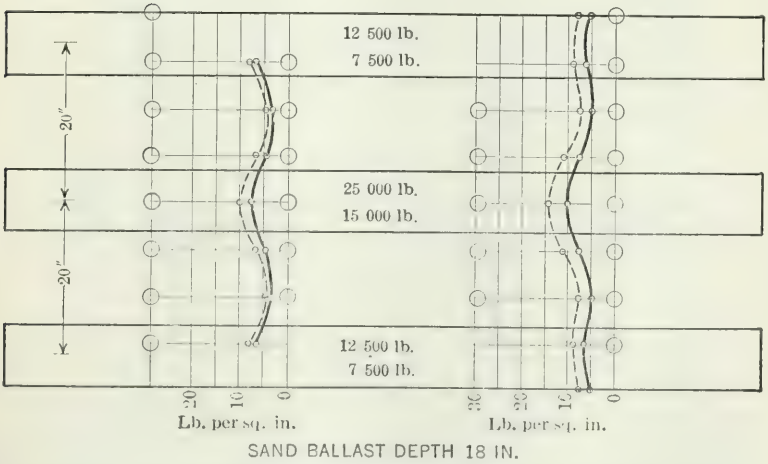
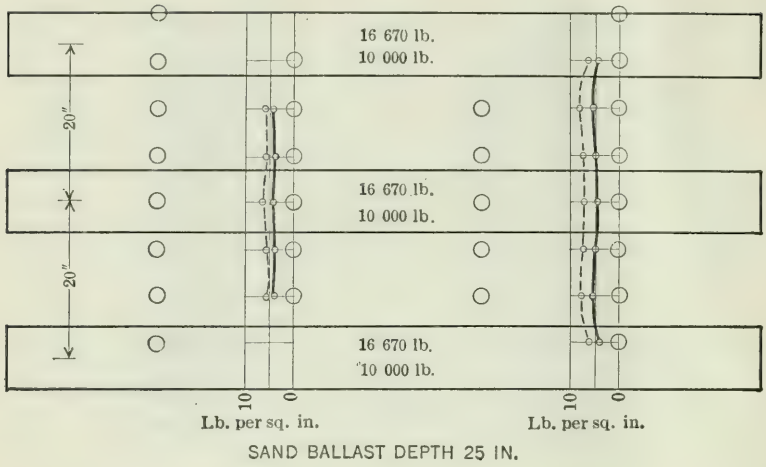


FIG. 82.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 18 IN. OF SAND BALLAST BELOW THREE TIES. EQUAL AND UNEQUAL LOADS ON TIES.



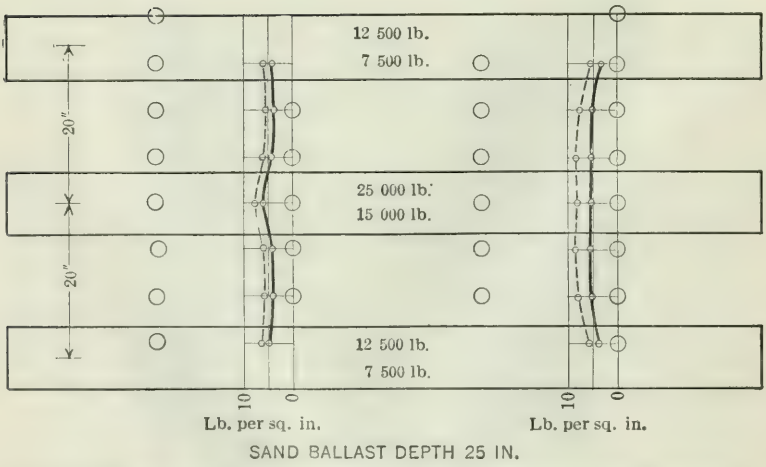


FIG. 83.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 25 IN. OF SAND BALLAST BELOW THREE TIES. EQUAL AND UNEQUAL LOADS ON TIES.

in. face horizontal, and a 6 by 8-in. by 8-ft. tie with the 6-in. face horizontal.

Fig. 86 shows the distribution of pressure through 18 in. of broken stone ballast when three loaded ties are used and also when all the load is carried on one tie.

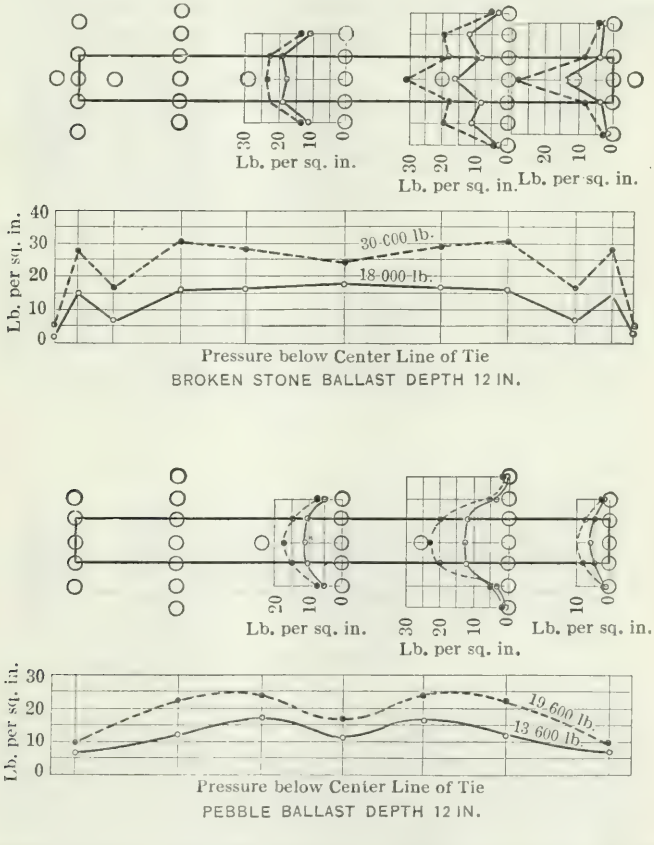


FIG. 84.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 12 IN. OF BROKEN STONE AND 12 IN. OF PEBBLE BALLAST BELOW A SINGLE TIE.

Fig. 87 shows the distribution of pressure on the bottom of a tie which rests on pebble ballast, and also the pressure measured simultaneously at a depth of 12 in. below the tie. The tie was prepared by embedding the capsules in the tie. The circular bearing plate of the capsule had been replaced with rectangular bearing plates, 3 by 1 in., and the surface of these was left flush with the bottom of the tie. The positions of the pressure capsules in the tie are indicated in the diagram.

C.—Discussion of Results of Tests.

40.—Diagrams of Average Pressure and Use of Superposition of Pressures.—In order to eliminate the variations in pressure due to flexibility of tie and other causes, for the study of the data, the pressures measured by all capsules located at the same distance from the longitudinal axis of the tie have been averaged. Using this method the diagrams in Fig. 88 have been drawn; they show the pressure at different depths and different distances from the center line of the tie

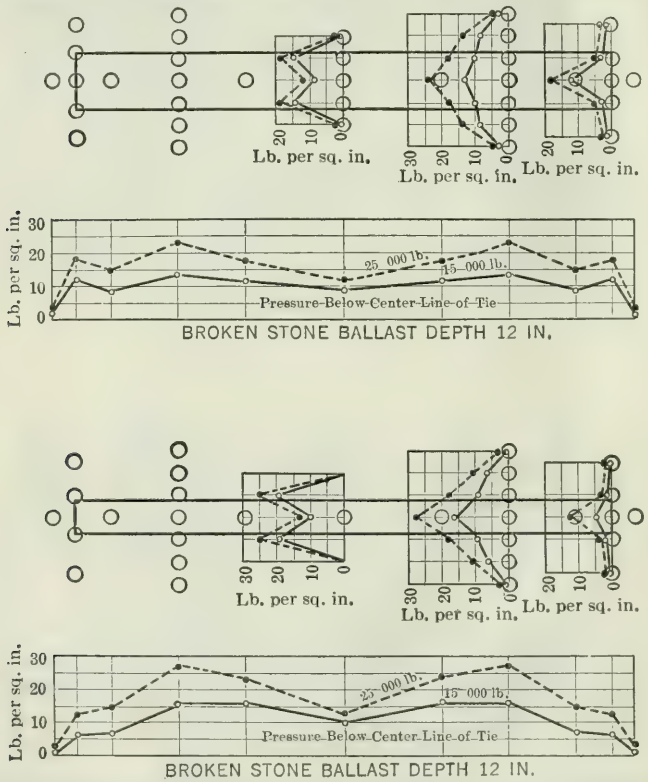

Lb. per sq. in.
Lb. per sq. in.
Lb. per sq. in.
Lb. per sq. in.
25 000 lb.
15 000 lb.
Pressure Below Center Line of Tie
BROKEN STONE BALLAST DEPTH 12 IN.
Lb. per sq. in.
Lb. per sq. in.
Lb. per sq. in.
Lb. per sq. in.
25 000 lb.
15 000 lb.
Pressure Below Center Line of Tie
BROKEN STONE BALLAST DEPTH 12 IN.

FIG. 85.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 12 IN. OF BROKEN STONE BALLAST BELOW A TIE 10 IN. WIDE AND BELOW ONE 6 IN. WIDE.

when a single tie is loaded, as given by the averages of the readings. The pressure is expressed as a percentage of the average intensity of pressure over the tie. The average result for the three loads has been used. In addition, the highest and the lowest pressure measured by any capsule is shown. Using the same method, the results obtained in tests with three ties have been plotted by full lines in Figs. 89 to 93. There have also been plotted the dotted lines marked "superposed pressure curve".

This method of superposition considers that at a given point the combined effect of several ties is the sum of the effects of the individual ties. The effect of a tie at a given point was obtained from the curves of Fig. 88. It will be noted that the superposed pressure curves agree

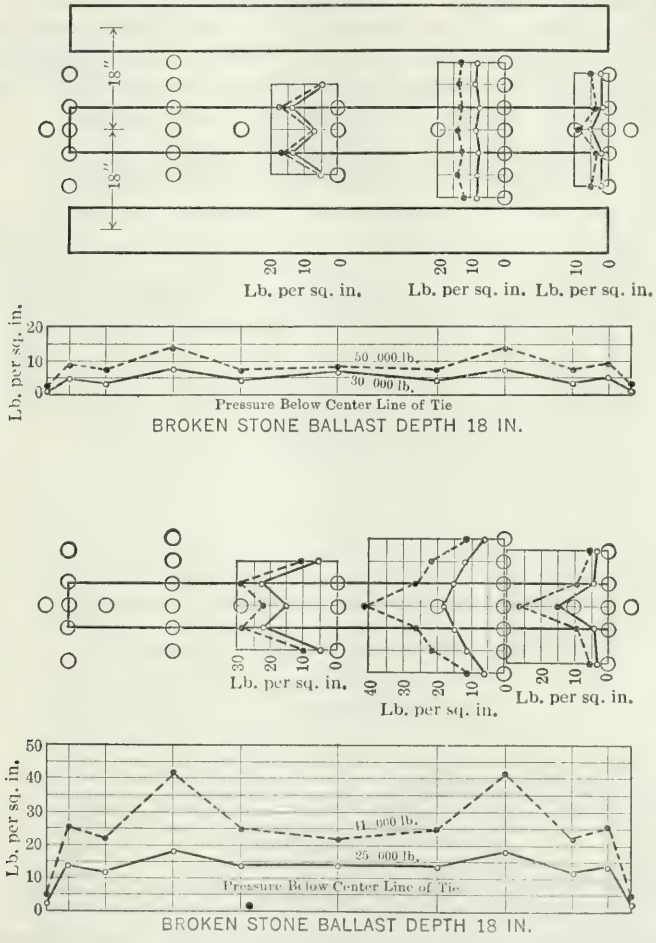


FIG. 86.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 18 IN. OF BROKEN STONE BALLAST BELOW THREE TIES AND BELOW A SINGLE TIE.

with the experimental curves reasonably well, indicating that the vertical pressure at any given point in the ballast is the sum of the vertical pressures which would be caused at that point by adding the pressures from the ties considered separately. By means of this principle, which seems to be established by these experiments, the results

of the experiments with a single tie may be used to determine the pressure in ballast under any condition of the spacing and loading.

41.—Comparison of Pressure Distributions in Different Kinds of Ballast.—An examination of the pressure curves for the sand, broken stone, and pebble ballasts will show that there are no decided differences in the distribution of the pressure at a given depth except that the results found with sand are more concordant than those with the coarser ballast. It may be concluded that the laws of distribution of pressure

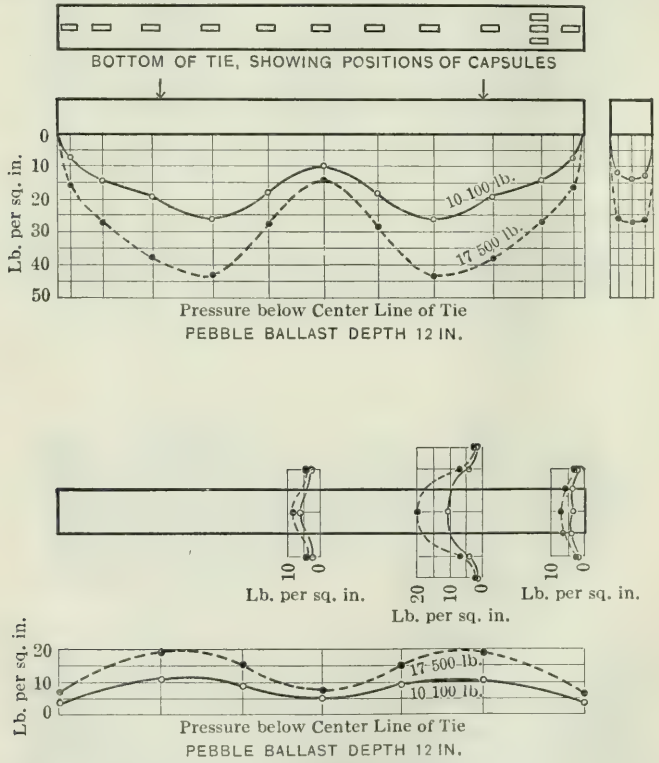


FIG. 87.—DISTRIBUTION OF PRESSURE AT A DEPTH OF 12 IN. OF PEBBLE BALLAST BELOW A SINGLE TIE AND AT THE BOTTOM OF THE TIE.

through ballast (non-cohesive granular material) are practically the same for the various kinds of ballast.

It should be borne in mind that the foregoing statement applies to distribution of pressure only; ultimate bearing load, action under vibration, and other properties of ballast will differ materially.

42.—Intensity of Pressure at Different Depths Below Center Line of Tie.—The intensity of pressure below the center line of the tie at various depths has been given in Fig. 88. Using values taken from this

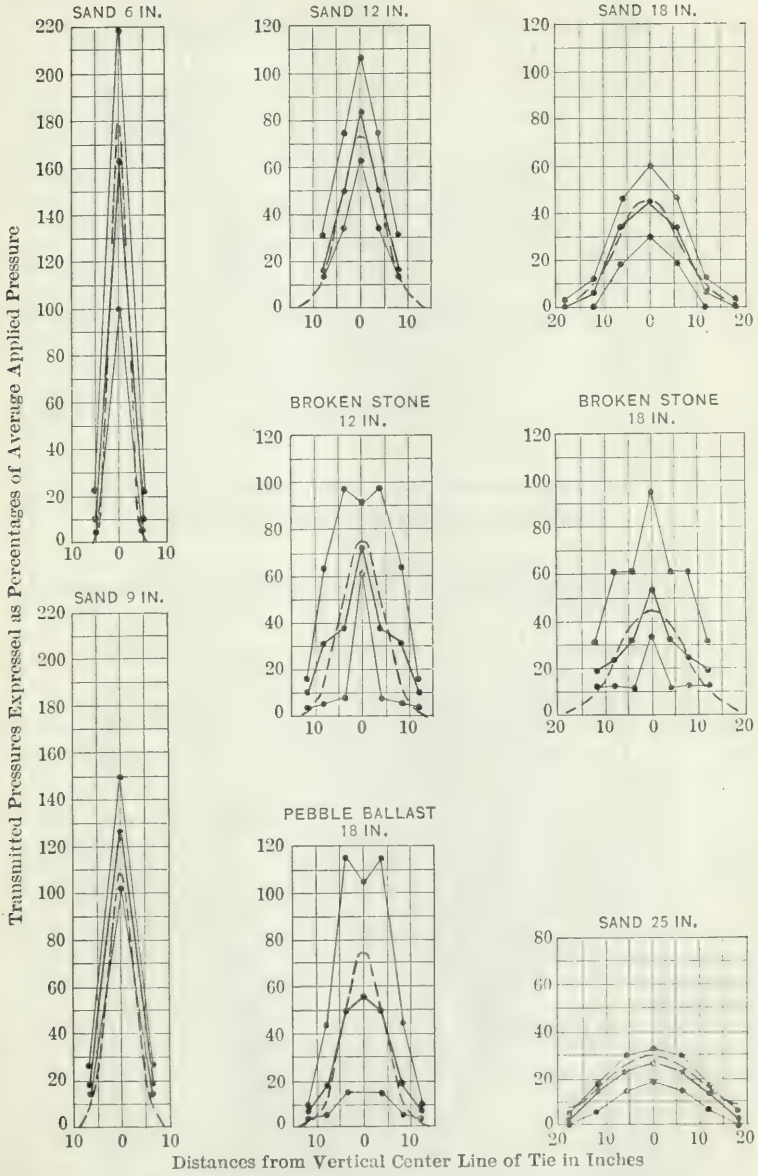

SAND 6 IN.
SAND 12 IN.
SAND 18 IN.
BROKEN STONE 12 IN.
BROKEN STONE 18 IN.
SAND 9 IN.
PEBBLE BALLAST 18 IN.
SAND 25 IN.
Transmitted Pressures Expressed as Percentages of Average Applied Pressure
Distances from Vertical Center Line of Tie in Inches

FIG. 88.—LATERAL DISTRIBUTION OF PRESSURE AT DEPTHS OF 6, 9, 12, 18, AND 25 IN. OF SAND BALLAST BELOW A SINGLE TIE. AVERAGE RESULTS OF TESTS.

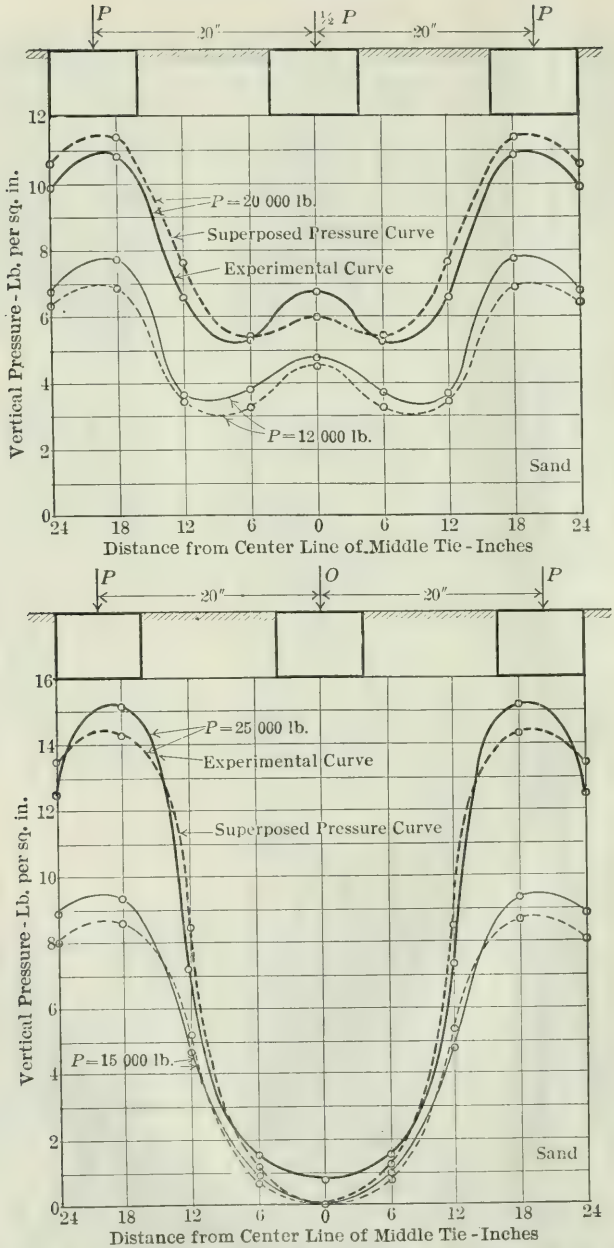


FIG. 89.—LATERAL DISTRIBUTION OF PRESSURE AT A DEPTH OF 18 IN. OF SAND BALLAST BELOW THREE TIES AND VALUES OBTAINED BY SUPERPOSING RESULTS FOUND WITH A SINGLE TIE.

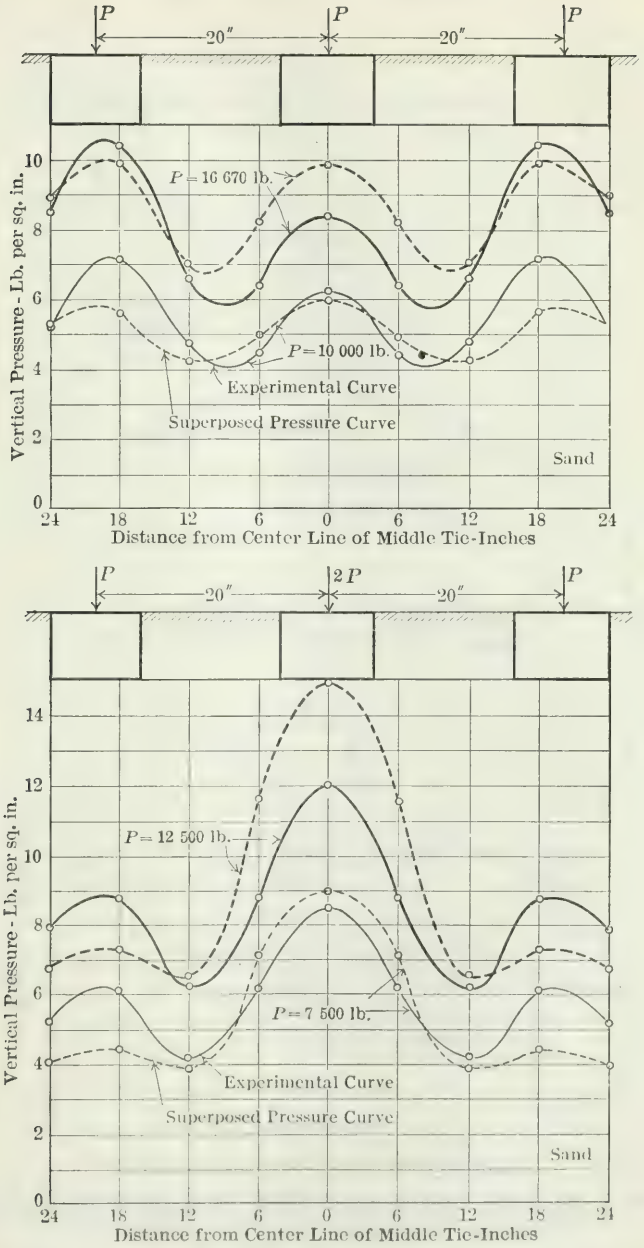


FIG. 90.—LATERAL DISTRIBUTION OF PRESSURE AT A DEPTH OF 18 IN. OF SAND BALLAST BELOW THREE TIES AND VALUES OBTAINED BY SUPERPOSING RESULTS FOUND WITH A SINGLE TIE.

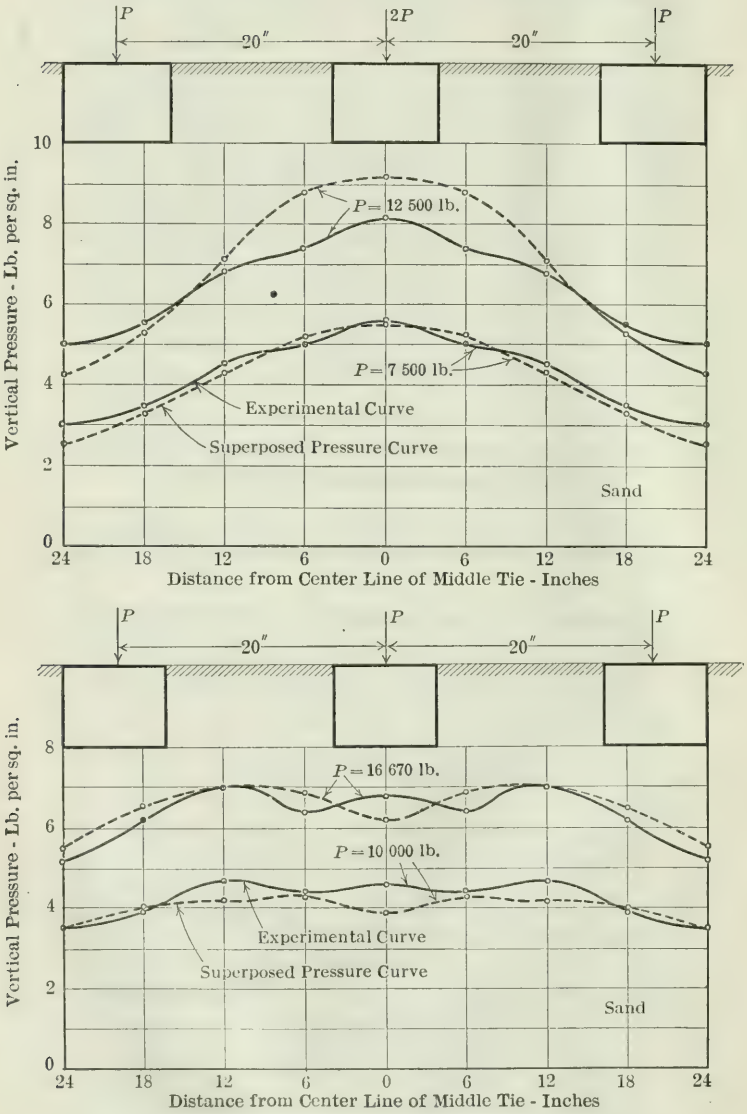


FIG. 91.—LATERAL DISTRIBUTION OF PRESSURE AT A DEPTH OF 25 IN. OF SAND BALLAST BELOW THREE TIES AND VALUES OBTAINED BY SUPERPOSING RESULTS FOUND WITH A SINGLE TIE.

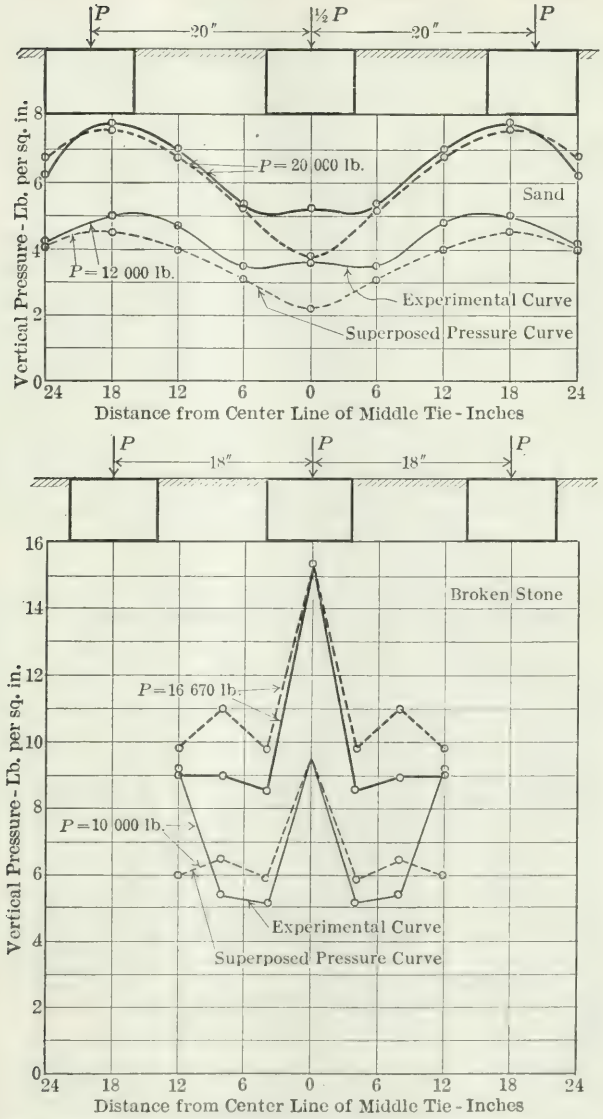


FIG. 92.—LATERAL DISTRIBUTION OF PRESSURE AT A DEPTH OF 25 IN. OF SAND BALLAST AND AT A DEPTH OF 12 IN. OF BROKEN STONE BALLAST BELOW THREE TIES AND VALUES OBTAINED BY SUPERPOSING RESULTS FOUND WITH A SINGLE TIE.

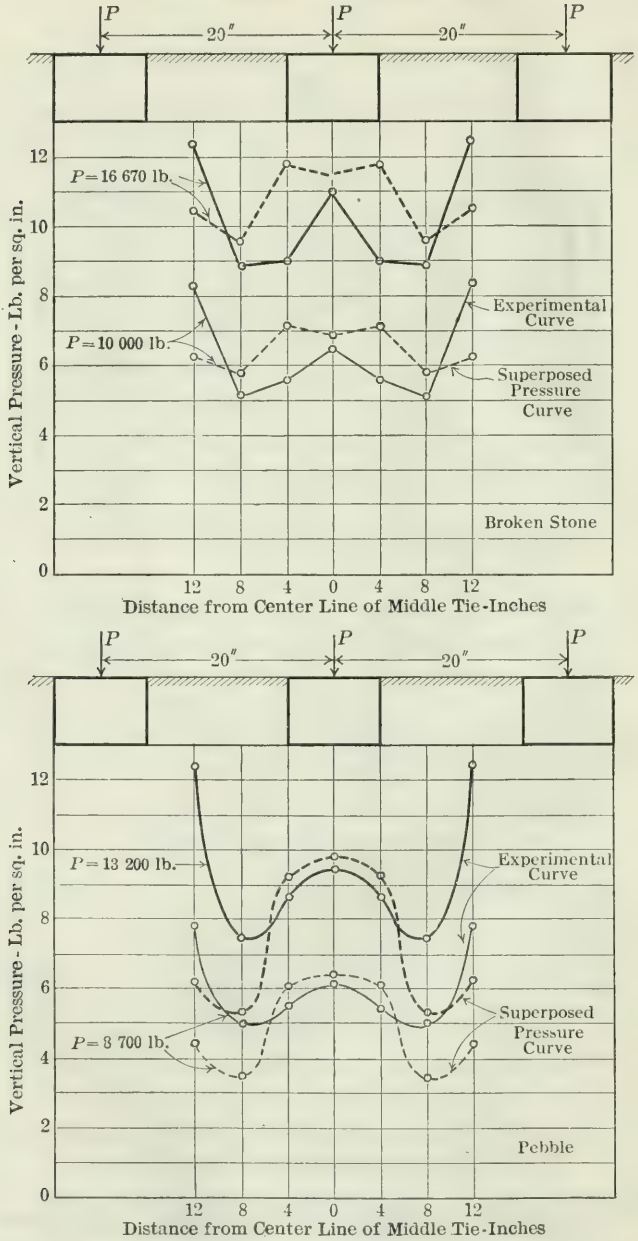


FIG. 93.—LATERAL DISTRIBUTION OF PRESSURE AT A DEPTH OF 18 IN. OF BROKEN STONE BALLAST AND OF 12 IN. OF PEBBLE BALLAST BELOW THREE TIES AND VALUES OBTAINED BY SUPERPOSING RESULTS FOUND WITH A SINGLE TIE.

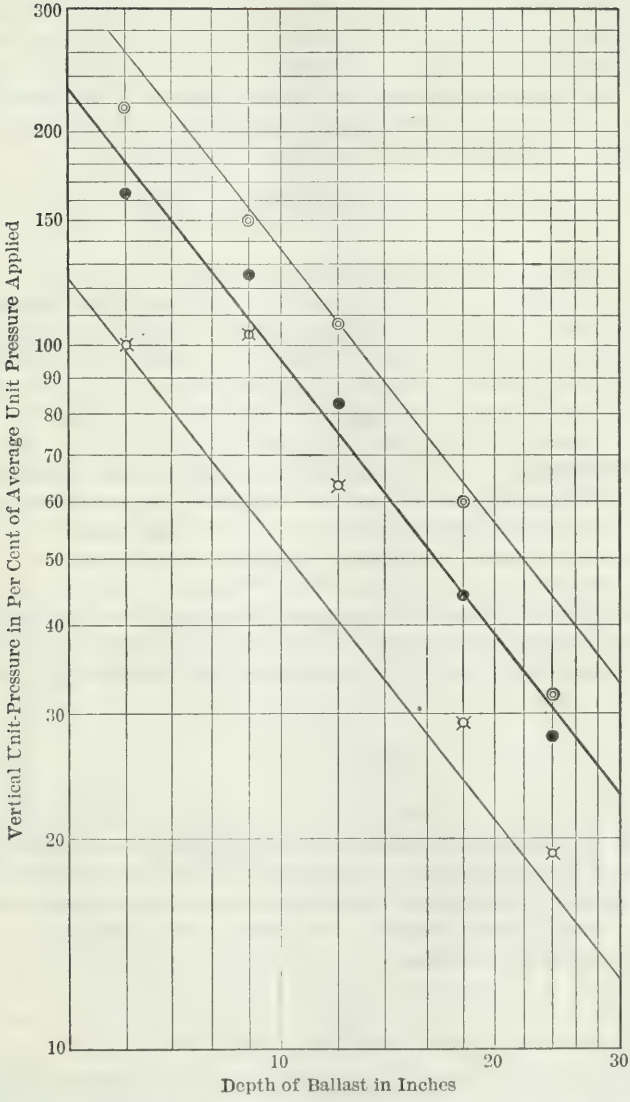


FIG. 94.—RELATION OF INTENSITY OF PRESSURE UNDER CENTER LINE OF TIE TO DEPTH BELOW BOTTOM OF TIE. LOGARITHMIC SCALE.

diagram, Fig. 94 has been drawn to express by logarithmic co-ordinates the relation between the unit-pressure at points below the center line of tie and the corresponding depth of ballast. The ordinate of a plotted point represents the pressure at a point below the center line of the tie expressed as a percentage of the average pressure over the bottom of the tie; the abscissa represents the depth of ballast at the point. The maximum and minimum observed intensities of pressure have also been plotted. A straight line the equation of which is

$$p_c = \frac{16.8\ p_a}{h^{1.25}} \qquad (25)$$

has been drawn and fits the observations fairly well within the range of the tests, although for depths less than 4 in. and more than 30 in. the results are too high. In this equation p_a is the average pressure over the tie and p_c the pressure at the given point directly below the center line of tie, both in pounds per square inch, and h is the vertical distance in inches below the bottom of the tie. It should be noted that this expression was obtained from tests with ties 8 in. wide.

43.—Distribution of Pressure Over a Horizontal Plane.—It is desired to obtain an equation for the intensity of pressure at any point (x, h) in the ballast, developed by a tie carrying load, which shall agree with the values of the pressure found by experiment.

From the discussion in Article 30 on "Paths of Pressure and Laws of Distribution of Pressure", it was concluded that the distribution of the intensity of vertical pressure on any horizontal plane due to a load p_a on a narrow strip would be represented by Equation (15) of the form

$$p = \frac{k\ p_a}{\sqrt{\pi}}\ e^{-k^2x^2}.$$

Since a tie may be considered to be made up of a number of narrow strips, the pressure in the ballast below a tie may be considered as a composite of the pressures due to the strips. It is found that the composite pressures in the ballast below a tie may also be represented by an equation of the form Equation (15) except at small depths. The equation may then be written

$$p = \frac{K\ P_a}{\sqrt{\pi}}\ e^{-K^2x^2} \qquad (26)$$

p being the intensity of vertical pressure on a horizontal plane at a distance x from the vertical center line of the tie due to a load p_a per inch of length of tie. For an 8-in. tie

$$P_a = 8\ p_a$$

in which p_a is the average pressure on bottom of tie in pounds per square inch. Hence,

$$p = \frac{8 K p_a}{\sqrt{\pi}} e^{-K^2 x^2}$$

When $x = 0$

$$p = p_c = \frac{8 K p_a}{\sqrt{\pi}}$$

In Article 42 it was found from the results of experiments with 8-in. ties in Equation (25) that

$$p_c = \frac{16.8\, p_a}{h^{1.25}}$$

Equating the values of p_c and solving for K:

$$K = \frac{2.1 \sqrt{\pi}}{h^{1.25}}$$

Substituting this value of K in the equation and changing from the base e to the base 10, the following empirical equation for p is obtained:

$$p = \frac{16.8\, p_a}{h^{1.25}} (10)^{-6.05 \frac{x^2}{h^{2.5}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

in which p_a is the average intensity of pressure over the tie per square inch, and p is the pressure per square inch at a point on a horizontal plane h in. below the tie and x in. from the center line of the tie. The equation applies only to an 8-in. tie and is applicable for depths from about 4 in. to about 30 in.

The dotted lines in Fig. 88 represent the pressures as determined by Equation (27). It will be seen that the agreement between the experiments and the equation is fairly good.

44.—Contours of Equal Vertical Unit-Pressure.—From the results of the tests it is possible to draw the contours of equal vertical unit-pressure beneath one or more loaded railroad ties. Fig. 95 shows the contours of pressure beneath a single 6 by 8-in. by 8-ft. tie. The contours are designated by percentages of the average pressure applied to the ballast by the tie. In Figs. 96, 97, and 98, contours of equal vertical pressure are shown beneath a series of equally loaded ties for three different tie spacings. It will be noted that in general the vertical pressures become nearly constant at a depth equal to the tie spacing. It should be added that in determining the contours the principle of superposition and the laws of distribution of pressure given by the equations already derived have been used. These laws may be applied to conditions not covered by the experimental work.

45.—Lines of Pressure.—In Article 30 on "Paths of Pressure and Law of Distribution of Pressure", it was concluded that a pressure p_a per unit length applied to a long narrow strip will develop an intensity

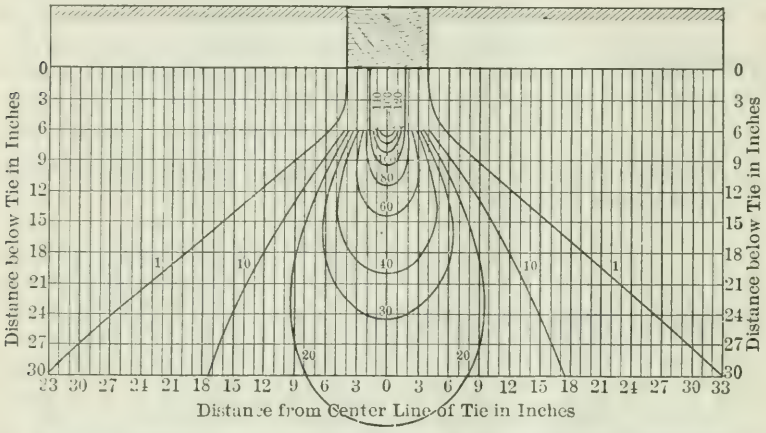


FIG. 95.—LINES OF EQUAL VERTICAL PRESSURE IN BALLAST FOR LOAD ON A SINGLE TIE.

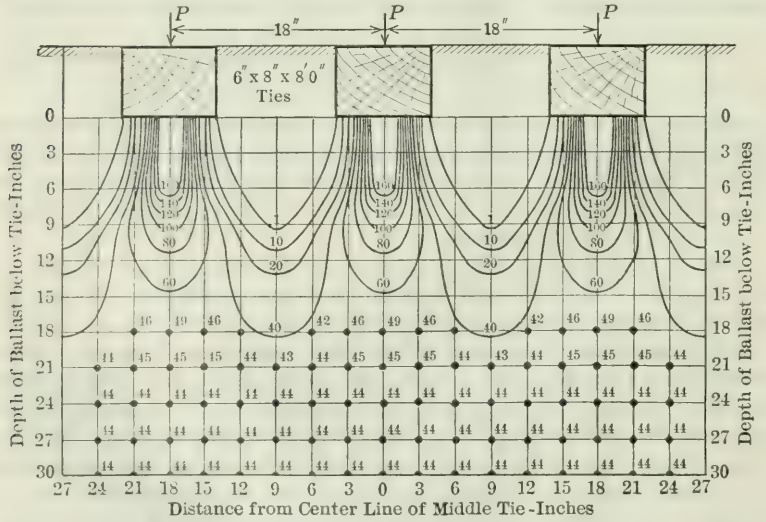


FIG. 96.—LINES OF EQUAL VERTICAL PRESSURE IN BALLAST FOR EQUAL LOADS ON TIES. SPACING 18 IN., CENTER TO CENTER.

of pressure p on a horizontal plane a distance h below the loaded strip and at a distance x from the line of action of the applied load p_a, in accordance with Equation (15) as follows:

$$p = \frac{k\,p_a}{\sqrt{\pi}}\,e^{-k^2x^2}.$$

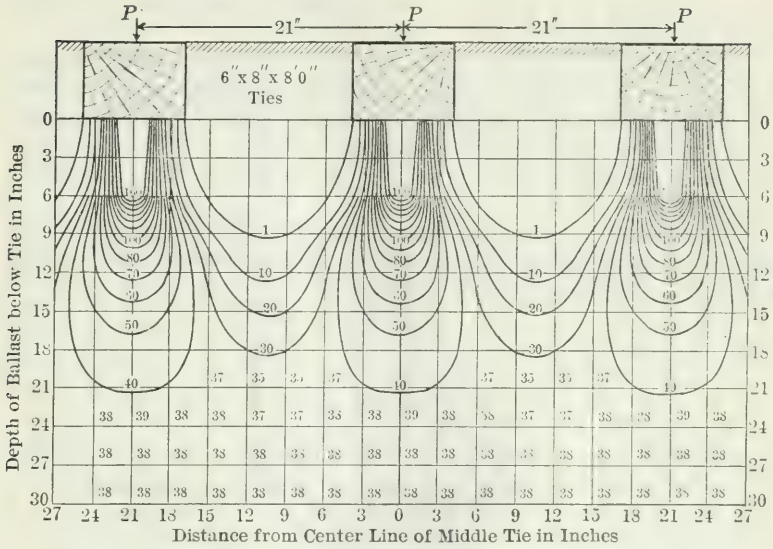


FIG. 97.—LINES OF EQUAL VERTICAL PRESSURE IN BALLAST FOR EQUAL LOADS ON TIES. SPACING 21 IN., CENTER TO CENTER.

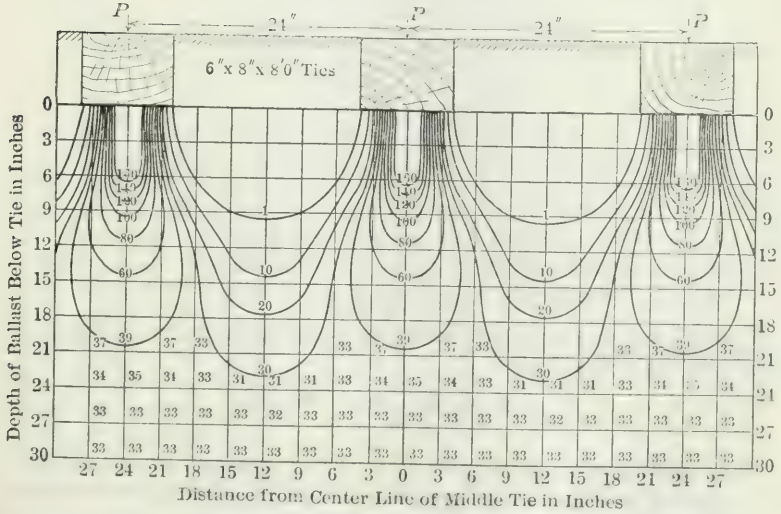


FIG. 98.—LINES OF EQUAL VERTICAL PRESSURE IN BALLAST FOR EQUAL LOADS ON TIES. SPACING 24 IN., CENTER TO CENTER.

In this equation k is a function of the depth h and must be determined by experiment.

The bottom of the tie may be considered to be divided into a number of narrow strips carrying loads of varying intensity and direction, each producing pressures in the ballast in accordance with the equation. At any point in the ballast the intensity of pressure will be equal to the sum of the pressures from the various strips into which the tie is divided.

In Article 31 it was shown that the distribution of pressure in the ballast due to a load on a narrow strip may be represented by lines which were designated lines of pressure.

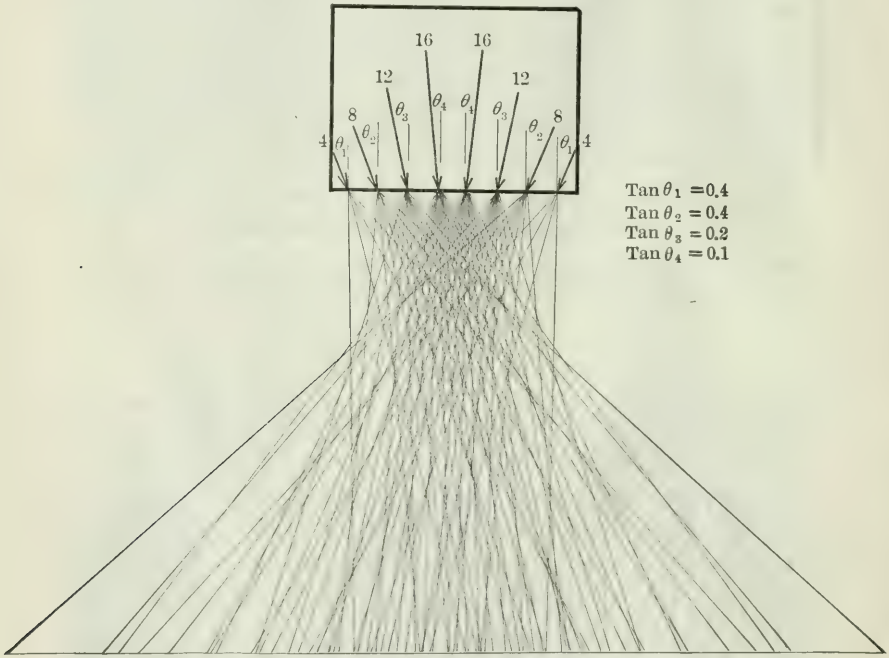


FIG. 99.—DIAGRAMMATIC REPRESENTATION OF LINES OF PRESSURE FOR A TIE 8 IN. WIDE.

After many trials it was found that for dimensions in inches Equation (15) may be written

$$p = 1.23 \frac{p_a}{h} (10)^{-2.06 \left(\frac{x}{h}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Taking the directions and intensities of the pressures p_a on the strips as indicated in Fig. 99, the pressures thus found in the ballast agree quite closely with the values determined in the experiments.

The lines of pressure determined from Equation (28) are straight lines, and were laid out in the following manner: The distribution of

pressure at a depth of 24 in. for the given value of p_a on a given strip was calculated by Equation (28). The results were then plotted on cross-section paper to a large scale using unit-pressures as ordinates and distances from the line of action of the applied pressure as abscissas. The area under the curve was divided into parts each representing 1 lb. and the abscissas of the centroids of such areas were determined. In making the desired division of the area it was found convenient to plot a mass curve beginning at the center line. From the center of the given strip on the tie, a line was drawn in the direction of the applied pressure p_a and a perpendicular line was drawn at a distance representing 24 in. Upon this perpendicular the abscissas determined previously were plotted. Straight lines through these points and the middle of the strip therefore represent lines of pressure of 1 lb. each. The number of such lines passing through a width of 1 in. on a horizontal plane represents the intensity of pressure.

TABLE 19.—Calculated and Experimental Values of Unit-Pressure at Various Points Below a Single Tie. Average Load on Tie, 100 Lb. per Sq. In.

h	x	Calculated p	Experimental p
6	0	161	163
9	0	106	126
12	0	75	83
12	4	47	50
12	8	12	16
18	0	50	45
18	6	30	34
18	10	15	16
18	15	5	4
24	0	37	31
24	10	18	17

Figs. 99 and 100 were plotted in the manner described, Fig. 99 representing the 1-lb. lines of pressure when the average load on the tie is 10 lb. per sq. in. and Fig. 100 when the average load is 100 lb. per sq. in., the only difference being that there are ten times as many lines in the last case. At any point in the ballast the number of lines in 1 in. of width will then represent the number of pounds of pressure per square inch at that point. As a line may represent any intensity of pressure (pounds being only as an illustration), both figures may be considered to represent the relative distribution of pressure for any given load on the tie.

The vertical unit-pressure at any point may be determined with greater accuracy by substituting for x and h in Equation (28) the values of x_1 and h_1, found by means of Equations (23) and (24) for each unit strip on the tie, and adding the results. The results

of some such calculations are given in Table 19, a width of tie of 8 in. being used, and experimental results are given for comparison.

It will be seen that the distribution of pressure found in the experiments may be accounted for by the superposition of pressures of differing intensities and inclinations over the bottom of the tie.

The method of lines of pressure, in conjunction with the variation of pressure beneath the tie, supplies explanations of the concentration and distribution of pressure in the ballast.

46.—General Discussion.—The following observations summarize some of the results of the tests and analysis and bring out in part the phenomena attending the transmission of pressure from the tie through the ballast.

The bearing pressure of the tie varies in intensity from its edge to its middle line; the maximum intensity is dependent upon the intensity of pressure developed at the edge. A variation in intensity exists also along the length of the tie.

The pressures which react from the lower face of the tie act in other than vertical lines, the greatest variation from the vertical direction being at the edge of the tie.

There is a concentration of pressure a short distance below the tie, say, at 3 to 4 in., and the intensity of pressure in the ballast at such a depth is greater than that which exists at the bottom of the tie.

For the tie of ordinary width the intensity of pressure at a depth of 6 in. and the distribution of vertical pressure over a horizontal plane at this depth do not differ greatly from those existing immediately under the tie. The directions of the pressures are not the same. At or below this depth the distribution of pressure laterally begins, with a consequent decrease in maximum intensity of pressure, and the change becomes more apparent as the depth increases.

The foregoing relates to the transmission of pressure from a single tie. For a number of ties with the ordinary tie spacing, the effect of the combination of pressures transmitted is readily found by superposing the values of the pressures from the several ties as obtained for a plane at the same depth. For the ordinary width of tie, the effect of the pressure transmitted from the adjacent tie to points midway between ties (overlapping lines of pressure) is noticeable at a depth equal to about half of the usual tie spacing. At a depth of three-fourths of the ordinary tie spacing the pressure immediately under the center of the tie is about one and one-half times that resulting from a uniform distribution over the horizontal plane. At a depth equal to the ordinary tie spacing the lateral distribution has become such that the variation in intensity of pressure from tie to tie is small.

The variation in intensity of pressure in the ballast lengthwise of the tie (which is dependent upon size and stiffness of tie, quality of tamping, and condition of the bed on which the tie rests) becomes

FIG. 100.—DIAGRAMMATIC REPRESENTATION OF LINES OF PRESSURE FOR A TIE 8 IN. WIDE, USING TEN TIMES AS MANY LINES AS ARE GIVEN IN FIG. 99.

less and less with increase in depth, and it may be expected that the variations will be smoothed out at a depth equal to the ordinary tie spacing, or a few inches below, where there will be a fairly uniform pressure over the horizontal plane.

The tests were made on a rigid base and the results may be expected to apply to a firm roadbed capable of carrying the loads transmitted. A depth of ballast greater than that named would be found useful when the roadbed is of uneven character or yields under the load or is subjected to unusually heavy load.

The tests show that for quiescent loading there is little difference in the manner and rate of transmission and distribution of pressure for broken stone, pebble, and sand ballasts; that is, at a given depth the intensities of pressure will be approximately the same, provided of course the ultimate carrying capacity of the ballast is not exceeded, and this conclusion may properly be extended to other non-cohesive materials. It will require less load to force the tie into sand ballast than into broken stone; the ultimate carrying capacity of the broken stone ballast under tie pressure is much greater than that of the sand ballast—the particles of sand ballast are more easily moved and rearrange themselves under lighter loads. For the different kinds of ballast there are great differences in the ultimate load which can be carried on a tie before ballast movement begins. The ultimate carrying capacity depends upon size of particle, smoothness of surface, and degree of angularity. A material the mobility of which under pressure is increased by the addition of water or by mixture with other materials may thereby have its carrying capacity decreased. For heavy loading the ultimate carrying capacity of a ballast material is especially important.

It is evident that a principal function of the ballast immediately under the tie and for some distance down, aside from such functions as drainage, is to carry the load without material lateral movement of the ballast to that depth at which lateral distribution becomes effective. An advantage of the coarser, rougher kinds of ballast is that they will carry a greater ultimate load—which is of special importance in the upper part of the ballast. This is especially true under the jarring, vibrating loads of track service for which the ultimate carrying capacity naturally will be less than that found under the quiescent loads used in the tests.

The tests in the laboratory indicate, as would be expected also from analytical considerations, that the presence of ballast above the level of the bottom of the tie may have little influence on the quiescent load which will be carried before the ballast will work out from under the tie and allow it to settle, but that under repeated applications of load and particularly under jarring and vibratory loads the ultimate carry-

ing capacity of the ballast is considerably increased by raising the level of the ballast surface to the top of the tie. This advantage is particularly apparent at the end of the tie, where under the whipping action of the tie under repeated deflections the particles of ballast will more readily be pushed away, since beyond the end of the tie there is no part of the track structure available for resisting the lateral pressure as is the case in the direction of the track, where another tie is always near at hand.

It seems probable that the effect of the jarring action of train loads will be to decrease the lateral distribution of pressure. It seems possible also that this tendency is counteracted in some degree by the cohesion which develops in ballast after it has been in place for some time.

It should be added that the "lines of pressure" used in the discussion are presented as being of service as an illustrative and suggestive conception which will help to explain some of the phenomena of transmission of pressure, rather than as a rigidly scientific hypothesis.

Respectfully submitted,

The Committee to Report on Stresses in Railroad Track

ARTHUR N. TALBOT, *Chairman,*
A. S. BALDWIN,
G. H. BREMNER,
JOHN BRUNNER
W. J. BURTON,
CHARLES S. CHURCHILL,
W. C. CUSHING,
W. M. DAWLEY,
ROBERT W. HUNT,
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PAUL M. LABACH,
C. G. E. LARSSON,
G. J. RAY,
ALBERT F. REICHMANN,
H. R. SAFFORD,
F. E. TURNEAURE,
J. E. WILLOUGHBY.

NOVEMBER 26TH, 1919.

DISCUSSION

A. W. Buel,* M. Am. Soc. C. E. (by letter).—The results of and deductions from the experiments to determine the distribution of pressure under railroad ties draws attention to the important influence of conjugate pressures or passive resistance in the ballast. Mr. Buel.

All the published tests were made with ties rectangular in cross-section—the almost universal standard in the United States. Ties of different cross-sections have been tried in at least two cases and perhaps in others. The writer refers to the triangular tie tested on the Great Northern Railroad and to the Percival reinforced concrete tie tried on the Southern Pacific System. Some seven or eight years ago his attention was called to the Percival tie, the moment of resistance of which he found by computation to be so small that the rather favorable report of the behavior of these ties in the track seemed to be almost incredible. It was stated, however, that, when alternated in the track with the standard rectangular tie, they did not last any length of time. It was reported by the division engineer in charge that this triangular tie could not be used when interspaced with standard ties, but acted very well when used with adjacent ties all of the same kind.

From a consideration of these facts the writer came to a tentative conclusion that the conjugate pressures or passive resistance of the ballast was the cause of the Percival tie holding up when, on the simple theory of its action as a beam, it should have broken. In other words, that the shape of the tie in cross-section changed the distribution of pressure in the ballast so as to reduce the bending moment on the tie.

An investigation of the influence of the shape of the tie on distribution of pressures certainly will develop some interesting results and possibly some very valuable ones. Six years ago the writer attempted to get some interested parties to finance such a series of experiments, and when this Special Committee was organized by the Society, he suggested to its Chairman that tests on ties of different cross-sections be included. The writer hopes that, before concluding its labors, the Committee will find a way to make such a series of tests and incorporate the results in future reports.

E. P. Goodrich,* M. Am. Soc. C. E. (by letter).—The writer has studied the Second Progress Report of the Special Committee to Report on Stresses in Railroad Track with considerable interest. Mr. Goodrich.

Several comments might be made with reference to this report, but the most important is in connection with the results discussed in Article 42, "Intensity of Pressure at Different Depths Below Center Line of Tie" (page 1562). An examination of Fig. 94 and a mathematical analy-

* New York City.

Mr. Goodrich.

sis of the results there plotted show that a semi-logarithmic plot would give a formula much nearer the apparent truth than that indicated in the report—in fact, the sum of the differences is reduced more than one-half. Vertical unit pressures should be plotted to a logarithmic scale, while the corresponding depth of ballast, in inches, should be plotted to an arithmetic scale. When this is done, the experimental results are found to fall substantially on straight lines. The equation of the average straight line thus secured, determined by the method of least squares, is,

$$\log. \frac{P}{P_a} = 2.471 - 0.0443\ h.$$

Obviously, this should be inserted in Equation (27) and in subsequent equations in place of the formulas adopted by the Special Committee. When this is done, results are obtained which are more concordant with fact.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1456

FINAL REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS*

WITH DISCUSSION BY MESSRS. HENRY S. PRICHARD, GUSTAV LINDENTHAL, CHARLES EVAN FOWLER, J. A. L. WADDELL, EDWARD GODFREY, WILLIAM H. BURR, R. VON FABRICE, L. J. MENSCH, AND THE STEEL COLUMN COMMITTEE.

The Special Committee, authorized by vote of this Society, "to consider and report upon the design, ultimate strength, and safe working values of steel columns and struts", was organized in January, 1909, with eleven members, as follows:

ALFRED P. BOLLER,	FRANK C. OSBORN,
AUSTIN L. BOWMAN,	GEORGE H. PEGRAM,
EMIL GERBER,	LEWIS D. RIGHTS,
CHARLES F. LOWETH,	GEORGE F. SWAIN,
RALPH MODJESKI,	EMIL SWENSSON,

J. R. WORCESTER.

Austin L. Bowman was elected Chairman, and Lewis D. Rights, Secretary.

Since that time, Alfred P. Boller, Emil Gerber, and Austin L. Bowman have died. They had brought ripe experience and enthusiastic interest to the work of the Committee, and its members desire at this time to record their appreciation of the invaluable work which these three gentlemen did while they were with us. James H. Edwards, Rudolph P. Miller, and C. W. Hudson were appointed to fill vacancies. Frank C. Osborn and Rudolph P. Miller have resigned from the Committee, on account of the pressure of other important duties. George H. Pegram, who served as Chairman after the death of Austin L. Bowman, resigned from the Committee upon being elected President of the Society, but has continued to take an interest in the work as

* Presented to the Annual Meeting, January 16th, 1918.

an *ex officio* member. Your Committee now consists of eight members, as will be seen by the list at the end of this report.

During 1909 the Committee collected literature on the subject of column tests, and tabulated and diagrammed the records of tests on columns with cross-sections of more than 5 sq. in. in area. This information was presented as a Progress Report to the Annual Meeting in 1910.* The report expressed the opinion that these data were not adequate, and that further tests were necessary.

During 1909 Congress authorized the construction of a large testing machine at Washington, and the Bureau of Standards, Dr. S. W. Stratton, Director, offered to make a series of tests on columns as soon as the machine was completed, the expense to be borne by the United States Government. The Committee and Dr. Stratton conferred as to the nature of the tests, and the Committee outlined the programme of a series of nine types of test columns, Figs. 1, 2, and 3. Owing to delays in constructing the machine, and difficulties in securing the special steel, the testing of the columns did not begin until January, 1914.

So as to provide a permanent record of the methods used, the Committee republishes below portions of the Progress Report of 1915, as follows:

Testing Machine and Instruments.

All the tests have been made at the Bureau of Standards, with an Emery testing machine, which was authorized by Congress on March 4th, 1909, and was accepted in 1913. Its capacity is as follows:

Tension	1 150 000 lb.
Compression	2 300 000 lb.
Extreme length between heads:	
Tension	34 ft. 4 in.
Compression	33 ft. 1¾ in.
Clearance between screws	4 ft. 6 in.
Length of stroke of ram	5 ft.

It will be seen from Fig. 4 that it has two heads, one fixed and the other movable, on two straining screws, each 12 in. in diameter. The hydraulic ram which furnishes the pressure is in the movable head, and, as the stroke is only 5 ft., it is necessary to adjust this head for columns of different lengths. The weighing is done from the fixed head of the machine.

The weighing device, which is peculiar to the Emery machines, consists of a specially constructed hydraulic support in the fixed head of the machine, which is connected with a smaller hydraulic chamber in the scale case. The load on the test specimen is transmitted directly to the oil in the hydraulic support, and by hydrostatic pressure is transmitted to the hydraulic chamber which actuates the levers

* *Transactions*, Am. Soc. C. E., Vol. LXVI (1910), p. 401.

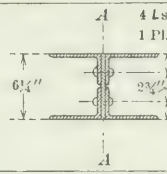
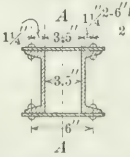
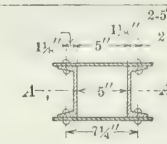
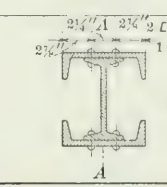
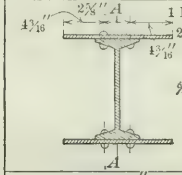
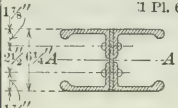
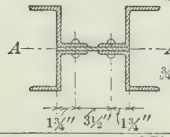
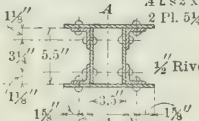
Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches	r	$\frac{l}{r}$	Length.
 4 Ls 5″ x 3″ x ⅜″ 1 Pl. 6″ x 5/16″	9.60 1.875 11.475	2.24	120 85 50	22′4 13/16″ 15′10 5/16″ 9′4″	4 Ls 5″ x 3″ x ⅝″ 1 Pl. 6″ x ⅝″ A Axis A ⅝″ Rivets	18.44 3.75 22.19	2.36	120 85 50	23′7 3/16″ 16′8 5/8″ 9′10″
 2-6″ Ls x 10.5-lb. 2 Pl. 8″ x ¼″ ⅞″ Rivets	6.18 4.00 10.18	2.31	120 85 50	23′1 3/16″ 16′4 3/8″ 9′7½″	2-6″ Ls x 15.5-lb. 2 Pl. 8″ x ½″ ½″ Rivets	9.12 8.00 17.12	2.33	120 85 50	23′3 5/8″ 16′6 1/16″ 9′8½″
 2-5″ Ls x 6.5-lb. 2 Pl. 9½″ x ¼″ ½″ Rivets	3.90 4.75 8.65	2.34	120 85 50	23′4 13/16″ 16′6 1/4″ 9′9″	2 Ls 5″ x 11.5-lb. 2 Pl. 9½″ x ½″ ½″ Rivets	6.76 9.50 16.26	2.39	120 85 50	23′10 13/16″ 16′11 1/4″ 9′11 1/4″
 2 Ls 8″ x 11¼-lb. 1 I 8″ x 18.0-lb ⅝″ Rivets	6.70 5.33 12.03	2.38	120 85 50	23′9¾″ 16′10 5/16″ 9′11″	2 Ls 8″ x 18.75-lb. 1 I 8″ x 20.50-lb. ⅝″ Rivets	11.02 6.03 17.05	2.32	120 85 50	23′2 1/16″ 16′5 3/16″ 9′8″

FIG. 1.

Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.
	7.37 6.88 14.25	2.31	120 85 50	23′1³/₁₆″ 16′4⅜″ 9′7½″	1 I 10″ x 35-lb. 2 Pl. 11″ x ⅝″ ⅝″ Rivets	10.29 13.75 24.04	2.47	120 85 50	24′8³/₈″ 17′5¹⁵/₁₆″ 10′3½″
4-5″ Bulb L^s x 10.1-lb. 1 Pl. 6″ x 5/16″ 	11.88 1.88 13.76	2.48	120 85 50	24′9⅝″ 17′6¹³/₁₆″ 10′4″	4-5″ Bulb L^s x 13.5-lb. 1 Pl. 6″ x 7/16″ ⅝″ Rivets	15.88 2.63 18.51	2.42	120 85 50	24′2⅜″ 17′1¹¹/₁₆″ 10′1″
4-4″ Z's x ¼″ 1 Pl. 7″ x ¼″ 	9.64 1.75 11.39	2.46	120 85 50	24′7⅜₁₆″ 17′5⅛″ 10′3″	4-4″ Z's x ⅜″ 1 Pl. 7″ x ⅜″ ¾″ Rivets	22.20 4.38 26.58	2.51	120 85 50	25′1³/₁₆″ 17′9³/₈″ 10′5½″
2 Pl. 9″ x ¼″ 4 L s 2″ x 2″ x ¼″ 2 Pl. 5¼″ x ¼″ 	4.500 3.760 2.625 10.885	2.33	120 85 50	23′3⅝″ 16′6¹/₁₆″ 9′8½″	2 Pl. 9″ x 7/16″ 4 L s 2″ x 2″ x 7/16″ 2 Pl. 5¼″ x 7/16″ ½″ Rivets	7.88 6.24 4.59 18.71	2.34	120 85 50	23′4¹³/₁₆″ 16′6⅞″ 9′9″

General Notes:
Rivet Spacing = 4 diameters of rivet
End distance = 2 " " "
Make three columns of each length for all shapes

FIG. 2.

Column and section.		Area □″	r	$\frac{l}{r}$	Length. Ft.	Length. Ins.
Light Section.	Bethlehem. 8-in. H-Column. 8 in. by 31.5 lb.	9.17	1.98	120	19	$9\frac{5}{8}$
		9.17	1.98	85	14	$0\frac{5}{16}$
		9.17	1.98	50	8	3
Heavy Section.	Bethlehem. 8-in. H-Column. 8 in. by 62 lb.	18.27	2.09	120	20	$10\frac{13}{16}$
		18.27	2.09	85	14	$9\frac{5}{8}$
		18.27	2.09	50	8	$8\frac{1}{2}$

Fig. 3.

of the weighing scale. These levers are all of the plate-fulcra type, characteristic of all Emery testing machines.

The construction of the hydraulic support and chamber is such that a movement of 0.01 in. of the brass diaphragms is sufficient to register the full capacity of the machine. By use of these diaphragms and guiding rings, friction of the parts is eliminated.

The programme calls for all the tests to be made with square-ended columns, and this is done by providing cast-steel bearing blocks at each head, which bear directly against the columns. Considerable difficulty has been found in securing bearing blocks of sufficient accuracy, and it was necessary for the Bureau to go over all the blocks and dress them to a surface with a variation of not more than plus or minus 0.001 in.

For measuring the compressions on the longitudinal axis of the columns, compressometers were used, which consisted of a bar, fixed at one end to the column, and the other end connected with a micrometer gauge, which reads by estimate to ten-thousandths of an inch. This gauge was equipped with an electrical buzzer, which enabled the operator always to secure an equal contact. At the beginning of the series, wood bars were used, but, owing to the length of time taken in making a test, it was decided to substitute hollow steel bars, in order to avoid any possibility of changes due to moisture and temperature.

The local stresses were measured with a Berry strain gauge, which is made with a pair of Invar steel bars some 10 in. long, with a fixed point at one end and a movable point with a bell-crank at the other end. The horizontal arm is connected with an Ames dial gauge, which reads, with the multiplication of the bell crank, to 0.0002 in., and by estimating to 0.00002 in. The nominal length between the contact points of this strain gauge is 8 in.

The strain gauge measurement was made in the usual way, *viz.*, two holes were drilled, 8 in. apart, using a No. 55 drill, and then countersunk with a tool having an angle of about 110 degrees. This insures a good shoulder for the points on the strain gauge, which is highly essential for accurate work with this type of instrument. Experienced operators can secure very uniform readings within limits of plus or minus 0.0002 in.

Originally, the horizontal and vertical deflections of the column under test were measured with a piano wire, with a spring tension which was fastened to plugs set in each end of the column. Measurements were taken at the center by scales. It was found that when the ends of the column began to deflect, or warp, the local bends were transmitted to the wire, and this method was then improved by stretching the wire from a fixed frame below the column. The difficulty in securing accurate measurements from the wire to the column within limits of plus or minus 0.01 in. still remained, and it was finally overcome by abandoning the longitudinal wire entirely, and connecting the column at the center with two dial gauges reading to 0.001 in., which were supported on frames, independent of the column. Tension wires perpendicular to the longitudinal axis of the column transmitted the horizontal and vertical movements to the two gauges. Two operators were employed in manipulating the gauges, and, due to the length of time required to read these, some slight flow of metal may have taken place during the reading, but this was hardly sufficient to change the general result.

Test Columns.

The Progress Report of the Committee, of January 15th, 1913,* shows the column sections which the Committee proposed to use, together with the chemical and physical properties. The diagram of these sections and the properties are incorporated with the present report. In addition, it should be noted from this report that the programme was "arranged with the intention, as far as practicable, of eliminating all variables except that of form", and was subdivided so as to consider the effect of light and heavy sections. Three columns of each slenderness ratio, 50, 85, and 120 of $\frac{l}{r}$, were provided for

* *Proceedings*, Am. Soc. C. E., for February, 1913.

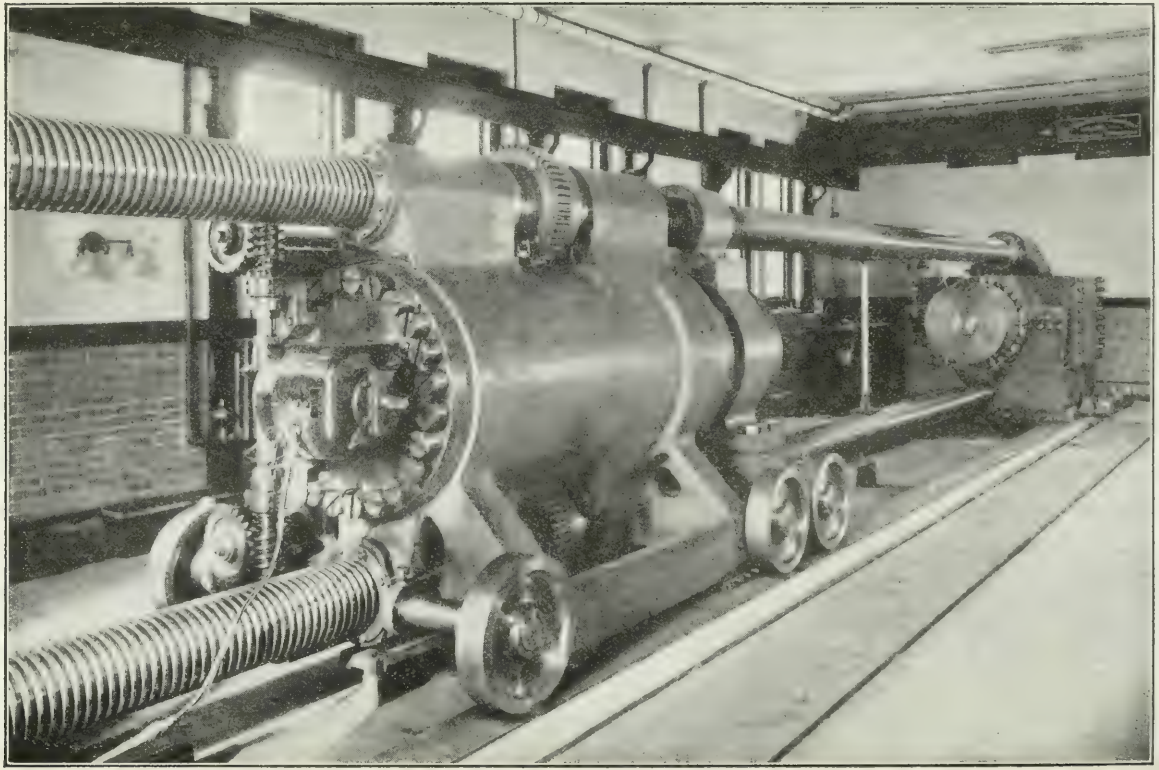


FIG. 4.—2 300 000-POUND TESTING MACHINE, BUREAU OF STANDARDS, WASHINGTON, D. C.

all shapes, making nine light sections and nine heavy sections, for each particular form of column, or a total of 144 test columns.

In addition to these, the Bureau of Standards has provided eighteen columns of Bethlehem H-sections, and eighteen columns of Carnegie 6-in. and 8-in. H-sections.

TABLE OF CHEMICAL AND PHYSICAL PROPERTIES FOR SPECIAL SPECIFICATIONS FOR UNIFORM QUALITY STRUCTURAL STEEL FOR COMPARATIVE COLUMN TESTS FOR THE BUREAU OF STANDARDS, U. S. GOVERNMENT.

Material.—Basic open-hearth structural steel.

Chemical.—Uniformity is paramount, and must be aimed for.

Carbon.—The percentage of carbon must not vary more than just to insure the desired physical properties in required sections of material.

Manganese	0.40% min. and 0.50% max.
Phosphorus	0.00% min. and 0.03% max.
Sulphur	0.00% min. and 0.04% max.
Silicon	0.00% min. and 0.10% max.
Nickel	0.00% min. and 0.05% max.
Chromium	0.00% min. and 0.05% max.
Copper	0.00% min. and 0.03% max.
All possible others	0.00%

Physical.—The desired physical properties must be aimed for.

Ultimate tensile strength	Desired, 60 000 lb. per sq. in. (may vary 1 500 lb. either way).
Elastic limit (yield point)	Desired, 38 000 lb. per sq. in. (may vary 1 000 lb. either way).
Elongation in 8 in.	Desired, 28% (may vary 2% either way).
Reduction in area	Desired, 56% (may vary 4% either way).
Character of fracture	Silky, cup.
Cold bend without fracture	180°, flat, on itself.

During the rolling of the material at the mill, and the fabrication at the shop, the inspectors of the Bureau of Standards investigated the various processes, making notes of the different conditions to which the material was subjected. Care was taken that the material met the rigid specifications, and although some variations were allowed, it was only with the special permission of the committee.

In addition to the material required for building the columns, the Bureau of Standards ordered additional pieces, 5 ft. long, of each shape and from each melt, so as to have an extra lot of unworked material on hand for special physical and chemical tests. The physical and

chemical tests were first made on all shapes by the manufacturer, and were then checked at the laboratory of the Bureau of Standards, at Pittsburgh.

It will be noted from the diagrams (Figs. 1, 2, and 3) showing the cross-sections of the columns that the rivets were all of standard size, the smallest being $\frac{1}{2}$ in. and the largest $\frac{3}{4}$ in. in diameter, and the rivet spacing called for four diameters throughout the total length of the columns. It was the intention of the Committee to over-rivet the sections, so as to insure the parts acting together as a whole. So far, in no case has a failure occurred due to a bulging of the material between rivets. Where the specimens have been compressed, in order to emphasize the method of failure, the rivets in all cases have remained tight.

In general, the shop work may be described as being uniformly good, and better than the average commercial grade. The ends were dressed square to the longitudinal axis of the column, but in almost all cases it was found necessary to re-dress them, owing to the fact that the ordinary commercial milling machine gives too rough a cut to insure complete bearing over the whole surface of the end cross-section of the column. After the test column was placed in the machine, special attention was given to insure a uniform bearing by applying loads of about 1 000 lb. per sq. in.

The investigation of the test columns developed variations in their sectional areas, these amounting, in extreme cases, to as much as 4% from the nominal area. In order to determine the correct area, the shapes were first calipered at the ends of the columns, and the variations noted. Prints of both ends were taken by blackening the ends and pressing white paper against them. The area was then determined by running a planimeter around the inside and outside perimeters. Finally, the column was weighed, the weight of the rivet heads was deducted, and the area was determined by the specific gravity of the material. This area, after being compared and checked with the results of the other two methods, was adopted as the correct one to be used in determining the unit loadings. When the column was put in the machine, before being tested, it was carefully examined for loose rivets, scale, or any other conditions which might affect its strength, and, during progress of the tests, rivets were tried and any scaling was noted. After the column was placed in the machine, a nominal load, of sufficient amount to insure its being held to the proper bearings, was applied, and the compressometers were attached. For the $\frac{l}{r}$ ratios of 50, 85, and 120, the test pieces ran on an average of 10, 16, and 23 ft. long, and the compressometers were arranged respectively with bars 50, 100, and 150 in. long. The two tests given*

* The details of these tests will be given later.

will indicate the details of applying the loads, and, as stated before, after each increment of 1 000 lb. per sq. in. had been applied, sufficient time was given to read the various strain gauges.

Failure of the column was first indicated by the dial gauges, which registered a rapid increase of the deflection at the center, and by the unbalancing of the scale beam, which showed a falling off of resistance to compression. With the exception of some additional scaling, the column at this time showed no apparent change, and the pressure could be maintained for several minutes before it had a perceptible bend.

As soon as the maximum loading was reached, the instruments were taken off, and one column of each series of three was further compressed, so as to emphasize the failure, and a photograph was taken to indicate how the column failed.

Methods of Testing.

After the material was ready for the beginning of the programme of testing, a conference was held between the physicists of the Bureau of Standards and a Sub-Committee of the Column Committee as to the methods of applying the loads and measuring the strains. A general outline of the instructions to the Engineer in Charge was agreed upon, and this outline is given below. Thus far it has not been necessary to make any marked changes in the methods. On certain tests variations have been introduced, at the request of the Committee, or at the suggestion of the physicist, but, thus far, these have not produced enough variation from the general result to warrant their continuance.

Outline of Instructions for Testing.—Columns shall be weighed and gauged for sizes before being tested. In addition, inspection shall be made to determine the condition of the riveting, in order to see that the pieces are properly drawn together, especial note being made of any local conditions where the riveting has failed to do this. Observations shall also be recorded where the pieces at the ends of the members have been bent. General alignment, waves in pieces or in the column as a whole, kinks, or any other special condition shall be noted.

The ends of the columns shall be dressed flat to reduce the amount of shimming to a minimum.

The two longer series of columns, that is, those of 85 and 120 $\frac{l}{r}$, shall be counterweighted at the middle by an amount equal to one-half of the total weight of the column.

Strain gauge measurements along the length of the column, at points from 8 to 10 in. apart, shall be made at the center of the column and near both ends. The gauge points at the ends shall be

placed far enough away to avoid effects of local distortion due to the taking up of the stresses. These strain gauge measurements shall be made at each of the four corners of the column and at the center of the webs and flanges. Special shapes, such as the Z-bar and bulb angle shape columns, may require variations from this general rule in order to determine the variation in stress in the different parts of the column.

Compressometer measurements shall also be made on longer lengths, of from 50 to 150 in., in order to check the short strain gauge measurements. Four of these will be made on each column—two on the corners and one on the center of the flange, and one on the center of the web.

Deflection observations shall be made on the horizontal and vertical movements at the middle of the length of each test piece.

The condition of the cross-section of the column shall be determined by calipering the two open sides before applying the initial load, and at intervals of loading of 5 000 lb. per sq. in., in order to determine any change in the shape of the cross-section. This shall be done in as many places as may be necessary.

Initial loads of 1 000 lb. per sq. in. each shall be applied, and compression loads of 5 000, 10 000, and 15 000 lb. per sq. in. each, returning to the initial load and determining permanent sets; from 15 000 lb. proceeding with increments of 1 000 lb. per sq. in. each, determining permanent sets after loads of 20 000 and 25 000 lb. per sq. in.; also at 30 000 lb. per sq. in., if these several loads are reached before the column fails.

Horizontal and vertical deflections of the columns shall be observed at the time the longitudinal compressions and sets are measured. The general direction taken by the column when it fails, and whether due to deflection as a whole or local buckling, shall be noted.

Careful observations shall be made where the column first begins to scale, also the lines of scale. After failure, record the conditions of the rivets around the distortion points to see if any of these rivets are loose.

After the ultimate load has been reached, at least one column of each cross-section and length shall be compressed, to emphasize the manner of failure.

In general, photographs shall be taken of one column of each cross-section and length when the ultimate load has been reached, and, in addition, a photograph of each column on which the failure has been emphasized.

Examples of Tests.

The Committee gives the record of two of the tests, *Nos. 30 and 54, which contain the information as furnished by the Bureau of Standards.

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* Plates XXXI and XXXII.

PLATE XXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1456.
FINAL REPORT OF
SPECIAL COMMITTEE ON
STEEL COLUMNS AND STRUTS.

Column Test Record
Form 260 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C., REPORT ON TEST No. 30

Date, April 3 and 4, 1914.

For whom tested · American Society of Civil Engineers.
Type No. or designation : 1.
Marks on column : 1 A C 5451.
Tested with flat ends.

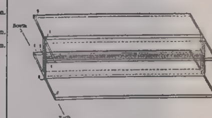
Counterweighted : one-half its weight at middle.
Nominal sectional area, in square inches : 11.475.
Radius of gyration : 2.24.
Slenderness ratio : 120.

Initial condition—
Riveting : Good.
Members at the ends : Good.
Alignment : Good (See note).

Length over all 22 ft. 4 11/16 in.
Weight in pounds · 898.
Sectional area, in square inches : 11 36. Corrected area, 11.20.
Gauged lengths : 150 and 8 in.

Applied Loads.		Compressions, in Inches, in 150-in. Gauged Lengths for Positions.					Mean compression in 150-in. gauged lengths (inches).	Deflections at Middle of Length. (Inches).		40 in. from Base. Compression, in Inches, in 8-in. Gauged Lengths at Base for Positions.								Straddling Middle. Compression, in Inches, in 8-in. Gauged Lengths at Middle for Positions.								Remarks.
Pounds, total.	*Pounds per square inch.	1	2	3	4	5		Horizontal. North.	Vertical. Down.	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8	
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	Temp. 20.9° cent.
56 800	5 000	0.0214	0.0212	0.0196	0.0205	0.0211	0.0208	0.00	0.00	0.0013	......	0.0013	0.0010	0.0011	......	0.0009	0.0011	0.0013	......	0.0012	......	0.0014	......	0.0012	0.0014	Note: 42 in. from top of column, had kink in leg of angle along line of gaugings No. 1 leg of angle bent in 0.6 in. The behavior of the angle is shown below.
11 360	1 000	0.0002	0.0006	0.0009	0.0004	0.0001	0.0001	....	......	0.0001	......	0.0001	0.0004	0.0001	......	0.0001	0.0000	0.0001	......	0.0000	......	0.0000	......	0.0001	0.0002	
56 800	5 000	0.0214	0.0212	0.0196	0.0206	0.0211	0.0208																			
113 600	10 000	0.0470	0.0465	0.0459	0.0471	0.0461	0.0465	0.00	0.00	0.0027	......	0.0027	0.0023	0.0026	......	0.0025	0.0025	0.0026	......	0.0026	......	0.0026	......	0.0028	0.0029	
11 360	1 000	0.0020	0.0021	0.0004	0.0024	0.0004	0.0011	....	......	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001	0.0001	......	0.0000	......	0.0000	......	0.0002	0.0004	
56 800	5 000	0.0224	0.0216	0.0191	0.0225	0.0186	0.0208																			
113 600	10 000	0.0469	0.0475	0.0449	0.0476	0.0446	0.0463																			
170 400	15 000	0.0722	0.0742	0.0716	0.0747	0.0730	0.0729	0.00	0.00	0.0042	......	0.0043	0.0037	0.0040	......	0.0040	0.0031	0.0041	......	0.0041	......	0.0043	......	0.0043	0.0044	Temp. 21° cent.
11 360	1 000	0.0022	0.0027	0.0015	0.0034	0.0011	0.0011	....	......	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001	0.0001	......	0.0000	......	0.0008	......	0.0004	0.0004	
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	April 4th. Temp. 19.5° cent.
56 800	5 000	0.0210	0.0208	0.0210	0.0210	0.0212	0.0210																			
113 600	10 000	0.0463	0.0466	0.0473	0.0473	0.0471	0.0468																			
170 400	15 000	0.0717	0.0723	0.0734	0.0732	0.0739	0.0729																			
181 760	16 000	0.0768	0.0783	0.0789	0.0787	0.0796	0.0784	0.00	0.00	0.0044	......	0.0045	0.0044	0.0043	......	0.0044	0.0043	0.0044	......	0.0044	......	0.0048	......	0.0044	0.0048	Snapping noise.
193 120	17 000	0.0812	0.0828	0.0841	0.0840	0.0848	0.0833	0.00	0.005 D	0.0047	......	0.0048	0.0048	0.0045	......	0.0047	0.0046	0.0047	......	0.0049	......	0.0049	......	0.0046	0.0049	
204 480	18 000	0.0850	0.0880	0.0892	0.0897	0.0901	0.0886	0.00	0.01	0.0050	......	0.0052	0.0052	0.0047	......	0.0050	0.0049	0.0050	......	0.0052	......	0.0051	......	0.0048	0.0052	
215 840	19 000	0.0911	0.0934	0.0949	0.0956	0.0960	0.0942	0.00	0.01	0.0053	......	0.0055	0.0055	0.0051	......	0.0054	0.0052	0.0052	......	0.0056	......	0.0055	......	0.0051	0.0055	Snapping noise.
227 200	20 000	0.0953	0.1008	0.1007	0.1014	0.1021	0.1001	0.00	0.02	0.0056	......	0.0058	0.0060	0.0054	......	0.0058	0.0054	0.0054	......	0.0059	......	0.0056	......	0.0056	0.0059	Slight drop of beam.
11 360	1 000	0.0003	0.0015	0.0010	0.0025	0.0013	0.0013	0.00	0.005	0.0001	......	0.0001	0.0003	0.0001	......	0.0000	0.0000	0.0000	......	0.0002	......	0.0000	......	0.0001	0.0003	Slight loosening of scale on angles.
56 800	5 000	0.0205	0.0212	0.0209	0.0223	0.0210	0.0212																			
113 600	10 000	0.0460	0.0473	0.0469	0.0486	0.0473	0.0472																			
170 400	15 000	0.0712	0.0735	0.0734	0.0750	0.0744	0.0735																			
227 200	20 000	0.0963	0.0999	0.1006	0.1016	0.1017	0.1000																			
238 560	21 000	0.1013	0.1056	0.1069	0.1077	0.1075	0.1058	0.00	0.02	0.0059	......	0.0061	0.0063	0.0056	......	0.0061	0.0059	0.0058	......	0.0063	......	0.0061	......	0.0059	0.0062	
249 920	22 000	0.1058	0.1121	0.1122	0.1141	0.1148	0.1117	0.00	0.02	0.0062	......	0.0065	0.0068	0.0061	......	0.0064	0.0061	0.0061	......	0.0063	......	0.0066	......	0.0061	0.0066	
261 280	23 000	0.1106	0.1184	0.1189	0.1212	0.1210	0.1180	0.00	0.03	0.0065	......	0.0070	0.0072	0.0064	......	0.0068	0.0065	0.0064	......	0.0072	......	0.0070	......	0.0066	0.0069	Snapping noise.
272 640	24 000	0.1151	0.1264	0.1267	0.1285	0.1293	0.1250	0.00	0.04	0.0067	......	0.0072	0.0076	0.0068	......	0.0072	0.0070	0.0066	......	0.0078	......	0.0074	......	0.0068	0.0072	
284 000	25 000	0.1188	0.1316	0.1354	0.1369	0.1368	0.1319	0.00	0.06	0.0071	......	0.0077	0.0080	0.0070	......	0.0076	0.0074	0.0068	......	0.0086	......	0.0078	......	0.0071	0.0075	Legs of angles bending.
11 360	1 000	0.0012	0.0057	0.0079	0.0096	0.0076	0.0069	0.00	0.06	0.0001	......	0.0006	0.0010	0.0001	......	0.0002	0.0002	0.0000	......	0.0011	......	0.0009	......	0.0008	0.0003	Scaling pretty general on 6-in. legs of angles.
56 800	5 000	0.0197	0.0278	0.0288	0.0299	0.0293	0.0270																			
113 600	10 000	0.0448	0.0528	0.0550	0.0568	0.0568	0.0580																			
170 400	15 000	0.0696	0.0798	0.0819	0.0839	0.0830	0.0793																			
227 200	20 000	0.0944	0.1055	0.1069	0.1106	0.1121	0.1063																			
284 000	25 000	0.1161	0.1338	0.1374	0.1381	0.1398	0.1330																			
318 700	28 055	......	......	......	......	......	......	0.30 N	1.00 D	Failed primarily by buckling of legs of angles, but finally by buckling in center. Temperature at finish 20.5° cent.																
318 700	28 455	Ultimate Strength According to Corrected Area.						....	......																	

Applied Loads.		40 in. from Top. Compression, in Inches, in 8-in. Gauged Lengths at Top for Positions.								Compression, in Inches, in ......Gauged Lengths.								Variations in Distance Between Legs of Angles, at Bend in Flange.								
Pounds, total.	*Pounds per square inch.	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8	Start of bend.	Max. bend or center.	Start of bend.	South side. 1/4	South side. 1/2	South side. 3/4	North side. 1/4	North side. 1/2	North side. 3/4
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	......	......	......	......	......	......	......	......	5.592	5.015	5.581						
56 800	5 000	0.0011	......	0.0009	0.0008	0.0012	......	0.0012	0.0012	......	......	......	......	......	......	......	......	5.591	5.008	5.585	0.000	......	0.000	0.000	......	0.002 open.
11 360	1 000	0.0000	......	0.0001	0.0001	0.0000	......	0.0001	0.0002	......	......	......	......	......	......	......	......	5.589	5.015	5.581						
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......						
113 600	10 000	0.0024	......	0.0022	0.0023	0.0026	......	0.0026	0.0030	......	......	......	......	......	......	......	......	5.600	4.993	5.588	......	......	......	......	......	0.004 open.
11 360	1 000	0.0001	......	0.0001	0.0001	0.0000	......	0.0001	0.0000	......	......	......	......	......	......	......	......	5.593	4.019	5.581						
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	5.606	4.985	5.594	......	......	......	......	......	0.006 open.
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......						
170 400	15 000	0.0035	......	0.0040	0.0036	0.0038	......	0.0037	0.0050	......	......	......	......	......	......	......	......	5.594	4.011	5.583						
11 360	1 000	0.0002	......	0.0000	0.0002	0.0000	......	0.0001	0.0001	......	......	......	......	......	......	......	......	......	......	......						
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	......	......	......	......	......	......	......	......	......	......	......						
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......						
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......						
170 400	15 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......						
181 760	16 000	0.0042	......	0.0043	0.0042	0.0040	......	0.0041	0.0063	......	......	......	......	......	......	......	......	5.606	4.974	5.596						
193 120	17 000	0.0044	......	0.0046	0.0044	0.0044	......	0.0043	0.0064	......	......	......	......	......	......	......	......	5.607	4.971	5.596						
204 480	18 000	0.0047	......	0.0050	0.0046	0.0047	......	0.0045	0.0071	......	......	......	......	......	......	......	......	......	......							
215 840	19 000	0.0050	......	0.0054	0.0048	0.0048	......	0.0046	0.0076	......	......	......	......	......	......	......	......	......	......							
227 200	20 000	0.0053	......	0.0058	0.0052	0.0050	......	0.0049	0.0082	......	......	......	......	......	......	......	......	......	......							
11 360	1 000	0.0000	......	0.0004	0.0000	0.0000	......	0.0004	0.0019	......	......	......	......	......	......	......	......	......	5.34							
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......							
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......							
170 400	15 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......							
227 200	20 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	5.19	......	......	......	......	......	......	0.01 open.
238 560	21 000	0.0056	......	0.0062	0.0055	0.0053	......	0.0049	0.0087	......	......	......	......	......	......	......	......	......	5.18							
249 920	22 000	0.0069	......	0.0068	0.0067	0.0065	......	0.0049	0.0094	......	......	......	......	......	......	......	......	......	5.17							
261 280	23 000	0.0062	......	0.0072	0.0060	0.0057	......	0.0051	0.0103	......	......	......	......	......	......	......	......	......	5.16							
272 640	24 000	0.0064	......	0.0077	0.0062	0.0058	......	0.0049	0.0113	......	......	......	......	......	......	......	......	......	5.14							
284 000	25 000	0.0068	......	0.0082	0.0064	0.0068	......	0.0046	0.0124	......	......	......	......	......	......	......	......	......	5.20							
11 360	1 000	0.0001	......	0.0011	0.0004	0.0004	......	0.0017	0.0044	......	......	......	......	......	......	......	......	......	5.01							



Test No. 30.

* Sectional area used in computing unit loading, 11.36 sq. in. Corrected area, 11.20 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 1.0143.

Details of Test No. 30, to Accompany Progress Report of Special Committee on Steel Columns and Struts.

TABLE 1.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES I & IA – I & IA SPECIAL.

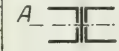
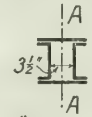
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	8	Light Sections	11.13	2.24	9'-4"	50	32400		0.035 S.	0.000
	38		11.17	"	"	"	32800	32700	0.07 S.	0.1000 Up.
	77		11.13	"	"	"	33000		0.050 N.	0.0313 Up.
	18		11.49	"	15'-10 7/16"	85	32100		0.12 S.	0.05 Up.
I	84		11.49	"	"	"	31400	31200	0.0252 S.	0.1281 Down.
	85	$4L^s\ 5"\times3"\times\frac{5}{16}" = 9.60^{a"}$ $1Pl.\ 6"\times\frac{5}{16}" = 1.875$ $Total = 11.475^{a"}$	11.35	"	"	"	30000		0.0626 N.	0.1368 Up.
	30		11.20	"	22'-4 13/16"	120	28600		0.000	0.060 Down.
	56		11.14	"	"	"	27600	28300	0.1175 S.	0.1091 Up.
	69		11.30	"	"	"	28700		0.0135 N.	0.1785 Down.
	93	Heavy Sections.	22.25	2.36	9'-10"	50	29000		0.0080 S.	0.0574 Down.
	96		22.13	"	"	"	28900	29200	0.0731 N.	0.0457 Up.
	97		22.28	"	"	"	29700		0.0653 N.	0.0002 Up.
	91		22.134	"	16'-8 5/8"	85	28000		0.0475 S.	0.0883 Up.
IA	94		21.92	"	"	"	28200	28100	0.4053 N.	0.2636 Up.
	95	$4L^s\ 5"\times3"\times\frac{5}{8}" = 18.44^{a"}$ $1Pl.\ 6"\times\frac{5}{8}" = 3.75$ $Total = 22.19^{a"}$	22.05	"	"	"	28000		0.0764 N.	0.0825 Down.
	92		22.15	"	23'-7 3/16"	120	25000		0.1430 S.	0.2923 Down.
	98		22.10	"	"	"	25500	25400	0.3863 N.	0.0762 Up.
	99		22.10	"	"	"	25800		0.1326 N.	0.0606 Down.
	85A	Light Sections	11.35	2.24	3'-8 3/4"	20	35100		0.0089 N.	0.0600 Down.
	84A		11.49	"	"	"	38000	36400	0.0230 S.	0.0313 Up.
	84B		11.49	"	"	"	36100		0.0141 S.	0.0120 Down.
I Spec.		as above.								
	99A	Heavy Sections	22.10	2.36	3'-11 3/16"	20	45000		0.0209 S.	0.0157 Down.
	98A		"	"	"	"	44100	44400	0.0177 N.	0.0104 Down.
	98B		"	"	"	"	44100		0.0031 N.	0.0229 Up.
IA Spec.		as above.								

TABLE 2.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 2 & 2A – 3 & 3A

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before failure	
2	11	Light Sections.	10.06	2.31	9'-7½"	50	33100		0.08 S.	0.00
	41		10.15	"	"	"	34000	33200	0.0024 N.	0.0374 Up.
	43		10.06	"	"	"	32400		0.0460 S.	0.0441 Up.
	21		9.92	"	16'-4⅜"	85	33700		0.12 S.	0.04 Up.
	59		9.87	"	"	"	32300	32600	0.1195 N.	0.0195 Up.
	65	2-6" [s x 10.5-lb = 6.18"	10.05	"	"	"	31700		0.0627 N.	0.0927 Down.
		2 Pls 8" x 1/4" = 4.00								
	32	Total = 10.18"	10.01	"	23'-1⅜"	120	28300		0.27 S.	0.12 Up.
	54		10.09	"	"	"	28700	29300	0.2750 S.	0.0583 Up.
	70		10.07	"	"	"	30900		0.1725 N.	0.0144 Up.
2A	101	Heavy Sections.	17.07	2.33	9'-8½"	50	32200		0.0318 N.	0.0360 Up.
	124		17.07	"	"	"	32500	32300	0.0052 N.	0.0044 Up.
	165		16.96	"	"	"	32300		0.1905 S.	0.0131 Up.
	145		17.07	"	16'-6⅛"	85	30700		0.0416 N.	0.0820 Down.
	146		17.02	"	"	"	30000	30600	0.0606 S.	0.0731 Up.
	151	2-6" [s x 15.5-lb = 9.12"	17.11	"	"	"	31000		0.0312 S.	0.0078 Down.
		2 Pls 8" x ½" = 8.00								
	135	Total = 17.12"	17.04	"	23'-3⅝"	120	28500		0.0668 S.	0.0298 Up.
	136		17.02	"	"	"	28800	28100	0.1315 S.	0.1942 Up.
	137		17.08	"	"	"	27000		0.1503 N.	0.0339 Down.
3	1	Light Sections.	8.71	2.34	9'-9"	50	33800		0.01 N.	0.06 Up.
	2		8.80	"	"	"	34300	34100	0.01 S.	0.04 Up.
	42		8.71	"	"	"	34300		0.0418 N.	0.0306 Up.
	14		8.71	"	16'-6 7/8"	85	32500		0.10 N.	0.06 Down.
	87		8.69	"	"	"	31900	32400	0.0764 S.	0.1994 Down.
	88	2-5" [s x 6.5-lb = 3.90"	8.60	"	"	"	32900		0.0470 S.	0.1774 Up.
		2 Pls 9" x 1/4" = 4.75								
	24	Total = 8.65"	8.79	"	23'-4 13/16"	120	30600		0.13 N.	0.01 Up.
	35		8.70	"	"	"	30200	30300	0.17 S.	0.08 Down.
	47		8.74	"	"	"	30000		0.2056 N.	0.1404 Down.
3A	105	Heavy Sections.	16.44	2.39	9'-11½"	50	29800		0.0084 N.	0.1044 Down.
	168		16.45	"	"	"	29300	29500	0.0077 N.	0.0170 Up.
	171		16.39	"	"	"	29500		0.0126 N.	0.0228 Up.
	147		16.41	"	16'-11⅛"	85	28000		0.0574 S.	0.0721 Down.
	148		16.39	"	"	"	28000	28000	0.0313 N.	0.0470 Down.
	150	2-5" [s x 11.5-lb = 6.76"	16.37	"	"	"	28000		0.1159 N.	0.0835 Up.
		2 Pls 9" x ½" = 9.50								
	138	Total = 16.26"	16.50	"	23'-10 13/16"	120	28000		0.1848 S.	0.0449 Down.
	139		16.45	"	"	"	26100	26900	0.0919 N.	0.3284 Down.
	143		16.42	"	"	"	26600		0.1201 N.	0.3920 Up.

COLUMN TEST RECORD
FORM 260 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C.—REPORT ON TEST No. 54.

Date, July 24, 1914.

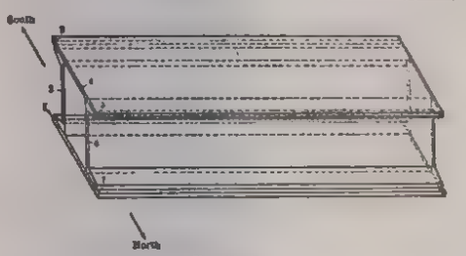
For whom tested: American Society of Civil Engineers.
Type No. or designation: 2.
Marks on column: 2A.
Tested with flat ends.

Counterweighted half its weight at middle.
Nominal sectional area, in square inches: 10.18.
Radius of gyration: 2.31.
Slenderness ratio: 120.

Initial condition—
Riveting: Good.
Members at the ends: Good.
Alignment: Vertical, good. Bent south 0.2 in. in center.

Length over all: 23 ft. 1-7/8 in.
Weight in pounds: 840.
Sectional area, in square inches: 9.66.
Ganged lengths: 150 and 8 in. Corrected area, 10.09 sq. in.

Applied Loads.		Compressions, in Inches, in 150-in Ganged Lengths for Positions.					Mean compression in 150-in ganged lengths (inches).	Deflections at Middle of Length (Inches).		Straddling Middle. Compressions, in Inches, in 8-in Ganged Lengths at Middle for Positions.								Remarks.
Pounds, total.	*Pounds per square inch.	1	3	5	7	4		Horizontal. South.	Vertical. Up.	1	2	3	4	5	6	7	8	
9 660	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	.....	0.0000	0.0000	0.0000	0.0000	
48 300	5 000	0.0197	0.0183	0.0204	0.0210	0.0198	0.0198	0.0097	0.0097	0.0012	0.0011	0.0010	.....	0.0012	0.0011	0.0011	0.0011	
9 670	1 000	0.0005	0.0002	0.0001	0.0002	0.0003	0.0001	0.0005	0.0134	0.0000	0.0000	0.0000	.....	0.0001	0.0000	0.0000	0.0001	
48 300	5 000	0.0198	0.0183	0.0202	0.0210	0.0212	0.0201											
96 600	10 000	0.0440	0.0420	0.0464	0.0477	0.0459	0.0452	0.0234	0.0172	0.0025	0.0024	0.0023	.....	0.0026	0.0025	0.0027	0.0025	
9 660	1 000	0.0011	0.0005	0.0004	0.0009	0.0020	0.0008	0.0016	0.0143	0.0000	0.0001	0.0000	.....	0.0001	0.0000	0.0000	0.0001	
48 300	5 000	0.0203	0.0188	0.0209	0.0205	0.0217	0.0216											
96 600	10 000	0.0443	0.0422	0.0469	0.0478	0.0463	0.0455											
144 900	15 000	0.0681	0.0656	0.0728	0.0745	0.0708	0.0704	0.0408	0.0214	0.0036	0.0035	0.0035	.....	0.0041	0.0039	0.0041	0.0039	
9 660	1 000	0.0013	0.0005	0.0004	0.0014	0.0023	0.0012	0.0030	0.0186	0.0000	0.0002	0.0000	.....	0.0002	0.0001	0.0002	0.0002	
48 300	5 000	0.0209	0.0193	0.0216	0.0220	0.0225	0.0214											
96 600	10 000	0.0446	0.0427	0.0472	0.0486	0.0469	0.0460											
144 900	15 000	0.0683	0.0657	0.0729	0.0747	0.0718	0.0706											
154 560	16 000	0.0729	0.0702	0.0784	0.0800	0.0763	0.0755	0.0446	0.0229	0.0040	0.0038	0.0037	.....	0.0044	0.0042	0.0044	0.0042	
164 220	17 000	0.0778	0.0747	0.0836	0.0853	0.0810	0.0805	0.0484	0.0235	0.0043	0.0041	0.0039	.....	0.0048	0.0046	0.0048	0.0045	Snapping noise.
173 880	18 000	0.0826	0.0793	0.0889	0.0906	0.0859	0.0855	0.0527	0.0245	0.0044	0.0044	0.0043	.....	0.0051	0.0048	0.0050	0.0048	
183 540	19 000	0.0873	0.0836	0.0940	0.0961	0.0908	0.0904	0.0575	0.0259	0.0047	0.0046	0.0044	.....	0.0053	0.0051	0.0054	0.0049	
193 200	20 000	0.0921	0.0883	0.0999	0.1020	0.0957	0.0956	0.0630	0.0272	0.0050	0.0050	0.0048	.....	0.0057	0.0054	0.0057	0.0053	
9 660	1 000	0.0018	0.0005	0.0011	0.0023	0.0028	0.0017	0.0061	0.0174	0.0001	0.0002	0.0001	.....	0.0001	0.0000	0.0001	0.0001	
48 300	5 000	0.0210	0.0192	0.0219	0.0234	0.0228	0.0216	0.0162	0.0152									
96 600	10 000	0.0450	0.0429	0.0481	0.0498	0.0473	0.0468	0.0297	0.0205									
144 900	15 000	0.0687	0.0653	0.0739	0.0755	0.0721	0.0711	0.0450	0.0242									
193 200	20 000	0.0923	0.0887	0.1000	0.1021	0.0962	0.0959	0.0637	0.0282									
193 200	20 000	0.0923	0.0889	0.1009	0.1022	0.0961	0.0960	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
193 200	20 000	0.0924	0.0891	0.1001	0.1022	0.0961	0.0960	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min.
193 200	20 000	0.0924	0.0892	0.1001	0.1022	0.0961	0.0961	0.0640	0.0282	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
202 860	21 000	0.0970	0.0934	0.1055	0.1077	0.1010	0.1009	0.0684	0.0289	0.0052	0.0052	0.0050	.....	0.0061	0.0058	0.0061	0.0056	
212 520	22 000	0.1015	0.0981	0.1111	0.1135	0.1063	0.1061	0.0752	0.0307	0.0056	0.0054	0.0052	.....	0.0064	0.0061	0.0064	0.0059	
222 180	23 000	0.1064	0.1025	0.1169	0.1192	0.1119	0.1112	0.0817	0.0324	0.0057	0.0058	0.0054	.....	0.0068	0.0064	0.0068	0.0063	Snapping noise
231 840	24 000	0.1118	0.1075	0.1231	0.1260	0.1168	0.1170	0.0905	0.0348	0.0060	0.0060	0.0056	.....	0.0070	0.0069	0.0072	0.0067	" "
241 500	25 000	0.1169	0.1121	0.1294	0.1327	0.1220	0.1226	0.0988	0.0378	0.0061	0.0062	0.0058	.....	0.0075	0.0072	0.0077	0.0071	
9 668	1 000	0.0039	0.0022	0.0019	0.0045	0.0049	0.0043	0.0226	0.0158	0.0001	0.0001	0.0001	.....	0.0001	0.0002	0.0002	0.0001	
48 300	5 000	0.0230	0.0209	0.0249	0.0274	0.0245	0.0241	0.0268	0.0204									
96 600	10 000	0.0470	0.0441	0.0512	0.0540	0.0493	0.0491	0.0407	0.0257									
144 900	15 000	0.0709	0.0677	0.0769	0.0800	0.0740	0.0739	0.0576	0.0300									
193 200	20 000	0.0943	0.0900	0.1033	0.1079	0.0986	0.0987	0.0783	0.0314									
241 500	25 000	0.1175	0.1124	0.1300	0.1337	0.1225	0.1232	0.1026	0.0385	.....	.....	.....	.....	.....	.....	.....	.....	Scale lines on top cover-plate 36 in. and 43 in. from base.
241 500	25 000	0.1176	0.1127	0.1304	0.1338	0.1226	0.1234	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
241 500	25 000	0.1176	0.1128	0.1304	0.1341	0.1227	0.1235	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min.
241 500	25 000	0.1178	0.1128	0.1306	0.1342	0.1228	0.1236	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
241 500	25 000	0.1179	0.1130	0.1306	0.1344	0.1230	0.1238	0.1041	0.0389	.....	.....	.....	.....	.....	.....	.....	.....	After 20 min.
241 500	25 000	0.1180	0.1130	0.1309	0.1344	0.1231	0.1239	0.1041	0.0392	.....	.....	.....	.....	.....	.....	.....	.....	After 25 min.
9 668	1 000	0.0044	0.0023	0.0044	0.0048	0.0052	0.0047	0.0172	0.0234	.....	.....	.....	.....	.....	.....	.....	.....	July 24th.
9 660	1 000	0.0020	0.0015	0.0029	0.0052	0.0038	0.0033	0.0171	0.0181	.....	.....	.....	.....	.....	.....	.....	.....	" 25th.
48 300	5 000	0.0224	0.0202	0.0236	0.0265	0.0236	0.0233	0.0261	0.0204									
96 600	10 000	0.0462	0.0435	0.0500	0.0529	0.0483	0.0482	0.0418	0.0258	.....	.....	.....	.....	.....	.....	.....	.....	Temperature: 27½° cent.
144 900	15 000	0.0701	0.0670	0.0759	0.0794	0.0729	0.0731	0.0591	0.0296									
193 200	20 000	0.0934	0.0897	0.1021	0.1069	0.0974	0.0977	0.0804	0.0344									
241 500	25 000	0.1167	0.1121	0.1292	0.1334	0.1204	0.1223	0.1042	0.0386									
241 500	25 000	0.1167	0.1119	0.1292	0.1344	0.1206	0.1224	0.1043	0.0385	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min. Scale lines on top cover-plate not any more developed.
251 160	26 000	0.1212	0.1163	0.1348	0.1390	0.1253	0.1273	0.1102	0.0396	0.0066	0.0067	0.0062	.....	0.0080	0.0078	0.0086	0.0075	
251 160	26 000	0.1214	0.1164	0.1349	0.1390	0.1255	0.1274	0.1111	0.0401	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
251 160	26 000	0.1215	0.1165	0.1349	0.1390	0.1254	0.1274	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min. Scaling a little more prominent.
251 160	26 000	0.1214	0.1164	0.1351	0.1393	0.1256	0.1275	0.1127	0.0401	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
251 160	26 000	0.1214	0.1162	0.1354	0.1395	0.1258	0.1277	0.1133	0.0401	.....	.....	.....	.....	.....	.....	.....	.....	After 20 min.
251 160	26 000	0.1216	0.1167	0.1354	0.1396	0.1258	0.1278	0.1135	0.0401	.....	.....	.....	.....	.....	.....	.....	.....	After 25 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 30 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 35 min.
260 820	27 000	0.1264	0.1208	0.1401	0.1449	0.1317	0.1340	0.1381	0.0431	0.0068	0.0069	0.0063	.....	0.0086	0.0084	0.0092	0.0080	
260 820	27 000	0.1264	0.1208	0.1404	0.1481	0.1320	0.1341	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
260 820	27 000	0.1263	0.1208	0.1442	0.1485	0.1322	0.1344	0.1371	0.0436	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min.
260 820	27 000	0.1266	0.1209	0.1439	0.1487	0.1324	0.1345	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
260 820	27 000	0.1267	0.1210	0.1442	0.1490	0.1727	0.1347	0.1405	0.0438	.....	.....	.....	.....	.....	.....	.....	.....	After 20 min.
260 820	27 000	0.1266	0.1205	0.1444	0.1493	0.1327	0.1347	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	After 25 min.
260 820	27 000	0.1267	0.1207	0.1446	0.1493	0.1329	0.1348	0.1421	0.0441	.....	.....	.....	.....	.....	.....	.....	.....	After 30 min.
270 480	28 000	0.1308	0.1247	0.1514	0.1588	0.1392	0.1407	0.1709	0.0477	0.0069	0.0072	0.0063	.....	0.0092	0.0090	0.0099	0.0083	
270 480	28 000	0.1311	0.1249	0.1531	0.1589	0.1395	0.1422	0.1869	0.0495	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
270 480	28 000	0.1312	0.1241	0.1552	0.1620	0.1400	0.1427	0.1910	0.0503	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min.
270 480	28 000	0.1309	0.1239	0.1555	0.1621	0.1400	0.1427	0.1946	0.0508	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
270 480	28 000	0.1311	0.1239	0.1569	0.1624	0.1403	0.1430	0.1970	0.0508	.....	.....	.....	.....	.....	.....	.....	.....	After 20 min.
270 480	28 000	0.1311	0.1239	0.1572	0.1633	0.1404	0.1432	0.1990	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 25 min.
270 480	28 000	0.1310	0.1239	0.1576	0.1637	0.1404	0.1433	0.2010	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 30 min.
270 480	28 000	0.1311	0.1239	0.1581	0.1638	0.1407	0.1435	0.2022	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 35 min.
270 480	28 000	0.1313	0.1239	0.1582	0.1638	0.1408	0.1436	0.2030	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 40 min.
270 480	28 000	0.1313	0.1239	0.1583	0.1640	0.1411	0.1437	0.2041	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 45 min.
270 480	28 000	0.1313	0.1239	0.1584	0.1640	0.1411	0.1437	0.2050	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 50 min.
270 480	28 000	0.1315	0.1240	0.1585	0.1643	0.1412	0.1439	0.2058	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 55 min.
270 480	28 000	0.1316	0.1240	0.1589	0.1645	0.1414	0.1441	0.2063	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 60 min.
270 480	28 000	0.1316	0.1241	0.1589	0.1648	0.1413	0.1441	0.2070	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 65 min.
270 480	28 000	0.1316	0.1241	0.1594	0.1652	0.1414	0.1443	0.2072	0.0509	.....	.....	.....	.....	.....	.....	.....	.....	After 70 min.
280 140	29 000	0.1353	0.1268	0.1689	0.1700	0.1482	0.1510	0.2750	0.0583	0.0068	0.0073	0.0066	.....	0.0105	0.0101	0.0116	0.0092	Scale lines at about 18 in. spacing on web of chan. on N. side.
280 140	29 000	0.1340	0.1245	0.1764	0.1842	0.1500	0.1538	0.3060	0.0615	.....	.....	.....	.....	.....	.....	.....	.....	After 5 min.
280 140	29 000	0.1338	0.1238	0.1786	0.1865	0.1507	0.1547	0.3237	0.0626	.....	.....	.....	.....	.....	.....	.....	.....	After 10 min.
280 140	29 000	0.1334	0.1232	0.1802	0.1883	0.1512	0.1553	0.3350	0.0633	.....	.....	.....	.....	.....	.....	.....	.....	After 15 min.
280 140	29 000	0.1333	0.1228	0.1810	0.1898	0.1516	0.1557	0.3457	0.0638	.....	.....	.....	.....	.....	.....	.....	.....	After 20 min.
280 140	29 000	0.1331	0.1225	0.1826	0.1910	0.1521	0.1563	0.3521	0.0642	.....	.....	.....	.....	.....	.....	.....	.....	After 25 min.
280 140	29 000	0.1332	0.1225	0.1839	0.1922	0.1524	0.1568	0.3605	0.0650	.....	.....	.....	.....	.....	.....	.....	.....	After 30 min.
280 140	29 000	0.1331	0.1225	0.1844	0.1927	0.1527	0.1571	0.3640	0.0655	.....	.....	.....	.....	.....	.....	.....	.....	After 35 min.
280 140	29 000	0.1331	0.1223	0.1850	0.1930	0.1528	0.1572	0.3670	0.0657	.....	.....	.....	.....	.....	.....	.....	.....	After 40 min.
280 140	29 000	0.1331	0.1222	0.1854	0.1935	0.1528	0.1574	0.3705	0.0662	.....	.....	.....	.....	.....	.....	.....	.....	After 45 min.
280 140	29 000	0.1331	0.1220	0.1854	0.1935	0.1529	0.1574	0.3717	0.0664	.....	.....	.....	.....	.....	.....	.....	.....	After 50 min.
289 800	30 000	0.1110	0.0925	0.2494	0.2658	0.1709	0.1779	0.9050	0.0566									
289 800	28 720	Ultimate Strength According to Corrected Area.																



Test No. 54.

* Sectional area used in computing unit loading, 9.66 sq. in. Corrected area, 10.09 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 0.9574.

TABLE 3.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 4 & 4A – 5 & 5A.

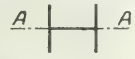
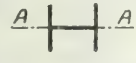
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
4	6	Light Sections.	11.69	2.38	9'-11"	50	35700		0.11 N.	0.015 Down.
	36		"	"	"	"	36500	36900	0.02 S.	0.04 Up.
	75		"	"	"	"	38600		0.0480 S.	0.0021 Up.
	13		11.66	"	16'-10 5/16"	85	34000		0.04	0.06 Up.
	79		11.73	"	"	"	33900	34000	0.1409 N.	0.0345 Up.
	80	2-8" ['s x 11.25-lb. = 6.70"	11.64	"	"	"	34000		0.0626 S.	0.1304 Down.
		1-8" I x 18.00-lb. = 5.33								
	26	Total = 12.03"	11.69	"	23'-9 5/8"	120	32000		0.15 S.	0.10
	45		11.61	"	"	"	30700	31900	0.409 N.	0.0216
	49		11.68	"	"	"	33100		0.0712 N.	0.0094 Up.
4A	102	Heavy Sections.	16.98	2.32	9'-8"	50	30300		0.0158 N.	0.0169 Down.
	166		16.84	"	"	"	29000	29100	0.0504 S.	0.0150 Down.
	169		16.81	"	"	"	28000		0.0311 S.	0.0019 Up.
	152		16.86	"	16'-5 3/16"	85	26000		0.1044 N.	0.0093 Up.
	153		16.89	"	"	"	26400	26600	0.2453 S.	0.0261 Down.
	154	2-8" [' s x 18.75-lb. = 11.02"	17.00	"	"	"	27500		0.0835 N.	0.0078 Down.
		1-8" I x 20.50-lb. = 6.03								
	132	Total = 17.05"	16.98	"	23'-2 7/16"	120	23500		0.4176 S.	0.0052 Down.
	133		17.07	"	"	"	23900	23900	0.2349 S.	0.0068 Up.
	134		16.96	"	"	"	24200		0.5283 N.	0.0616 Down.
5	107	Light Sections.	9.62	1.98	8'-3"	50	38000		0.0280 N.	0.0115 Down.
	122		9.65	"	"	"	38000	38000	0.0836 N.	0.0287 Down.
	123		9.65	"	"	"	38000		0.0052 S.	0.0209 Down.
	114		8.61	"	14'-0 5/16"	85	36000		0.1510 S.	0.0078 Down.
	116		9.66	"	"	"	34000	34300	0.1148 S.	0.0085 Up.
	119	Beth. 8" H Col x 32-lb.	9.59	"	"	"	33000		0.1201 S.	0.0376 Up.
		= 9.17"								
	110		8.64	"	19'-9 5/8"	120	33900		0.1148 N.	0.0084 Up.
	111		8.68	"	"	"	31000	32000	0.2140 N.	0.0136 Down.
	108		8.61	"	"	"	31000		0.1806 S.	0.0110 Up.
5A	106	Heavy Sections.	17.86	2.09	8'-8 1/2"	50	34000		0.1201 N.	0.0084 Up.
	120		17.82	"	"	"	36100	35400	0.0606 N.	0.0016 Up.
	121		17.86	"	"	"	36000		0.0284 N.	0.0033 Up.
	115		18.05	"	14'-9 5/8"	85	33900		0.0521 N.	0.0151 Down.
	117		17.73	"	"	"	32000	32300	0.0929 N.	0.0054 Down.
	118	Beth. 8" H Col x 62-lb.	17.77	"	"	"	31000		0.0470 S.	0.0277 Up.
		= 18.27"								
	109		17.76	"	20'-10 13/16"	120	30000		0.1498 N.	0.0037 Up.
	112		17.97	"	"	"	30000	30000	0.2401 S.	0.0219 Down.
	113		17.73	"	"	"	30000		0.1610 N.	0.0221 Down.

TABLE 4.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 6 & 6A—10 & 10A.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
6	10	Light Sections.	13.72	2.31	9'-7½"	50	31200		0.11 N.	0.01 Up.
	40	A	13.78	"	"	"	31500	31600	0.0837 S.	0.0065 Up.
	44		13.72	"	"	"	32200		0.0250 N.	0.0020 Up.
	22		13.65	"	16'-4 3/8"	85	27800		0.30 N.	0.02 Up.
	63	A	13.68	"	"	"	29500	29100	0.3040 N.	0.0097 Down.
	64	1-I 10"x25-lb.=7.37a"	13.60	"	"	"	30100		0.1306 S.	0.0166 Down.
		2 Pls. 11"x 5/16"=6.88								
	31	Total = 14.25a"	13.73	"	23'-1 3/16"	120	26700		0.50 S.	0.04 Up.
	52		13.75	"	"	"	27700	27200	0.1980 S.	0.0190 Down.
	71		13.65	"	"	"	27300		0.1369 S.	0.0013 Up.
6A	103	Heavy Sections.	23.98	2.47	10'-3½"	50	31200		0.0175 S.	0.0019 Down.
	125	A	23.75	"	"	"	32800	32100	0.0078 S.	0.0076 Up.
	128		23.72	"	"	"	32400		0.0496 N.	0.0042 Up.
	155		23.69	"	17'-5 15/16"	85	27200		0.3028 N.	0.0061 Up.
	158	A	23.73	"	"	"	26800	26800	0.3341 N.	0.0312 Down.
	159	1-I 10"x35-lb.=10.29a"	23.72	"	"	"	26400		0.3341 S.	0.0522 Down.
		2 Pls. 11"x 5/8"=13.75								
	129	Total = 24.04a"	23.64	"	24'-8 3/8"	120	24900		0.1827 S.	0.0097 Up.
	130		23.83	"	"	"	24000	24800	0.4100 S.	0.0157 Down.
	131		23.76	"	"	"	25500		0.3654 N.	0.0099 Up.
10	3	Light Sections.	10.77	2.33	9'-8½"	50	36100		0.06 S.	0.055 Up.
	4	A	10.80	"	"	"	35700	35800	0.05 N.	0.0251 Up.
	53		10.86	"	"	"	35600		0.0118 N.	0.0217 Up.
	19		10.78	"	16'-6 1/16"	85	32500		0.29 S.	0.200 Up.
	83	3½" A	10.76	"	"	"	31900	32100	0.0239 N.	0.1117 Down.
	86	2 Pls. 9"x 1/4"=4.500a"	10.76	"	"	"	31900		0.2611 N.	0.0786 Down.
		4 Ls. 2½"x2½"x 1/4"=3.760								
	23	2 Pls. 5 1/4"x 1/4"=2.625	10.75	"	23'-3 3/8"	120	28400		0.30 N.	0.05 Down
	33	Total = 10.885a"	10.76	"	"	"	28300	28400	0.23 S.	0.03 Up.
	34		10.75	"	"	"	28400		0.20 S.	0.00
10A	104	Heavy Sections.	18.60	2.34	9'-9"	50	31400		0.0360 S.	0.1556 Down.
	167	A	18.52	"	"	"	32000	31800	0.0099 N.	0.0851 Down.
	170		18.46	"	"	"	32100		0.0783 N.	0.0188 Down.
	144		18.67	"	16'-6 7/8"	85	28000		0.0292 S.	0.1044 Down.
	149	3½" A	18.65	"	"	"	28000	28300	0.1545 S.	0.0992 Down.
	157	2 Pls. 9" x 7/16"=7.88a"	18.69	"	"	"	29000		0.0731 N.	0.0010 Down.
		4 Ls. 2½"x2½"x 7/16"=6.24								
	140	2 Pls. 5 1/4"x 7/16"=4.59	18.64	"	23'-4 13/16"	120	27000		0.0392 N.	0.1080 Up.
	141	Total = 18.71a"	18.80	"	"	"	25900	26300	0.2260 S.	0.1112 Up.
	142		18.65	"	"	"	26000		0.2297 S.	0.1113 Up.

TABLE 5.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 7 & 7A – 8 & 8A

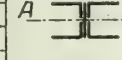
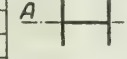
Type	Test no	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns, at the Reading just before Failure.	
7	5	Light Sections.	13.33	2.48	10′-4″	50	34300		0.00	0.03 Down.
	37		13.30	"	"	"	32700	33400	0.02 S.	0.04 Up.
	76		13.33	"	"	"	33200		0.0034 N.	0.1628 Down.
	15		13.40	"	17′-6 13/16″	85	34000		0.10 S.	0.40 Up.
	89		13.43	"	"	"	30000	31600	0.1081 N.	0.2923 Up.
	90	4-5″ Bulb Ls x 10.1-lb. = 11.88″	13.29	"	"	"	30700		0.0914 N.	0.1670 Up.
		1 Pl. 6″ x 5/16″ = 1.88								
	28	Total = 13.76″	13.36	"	24′-9 5/8″	120	28300		0.05 S.	0.18 Up.
	48		13.34	"	"	"	26900	28100	0.0137 N.	0.3020 Down.
	51		13.33	"	"	"	29000		0.0020 N.	0.0160 Up.
7A*		Heavy Sections.								
		4-5″ Bulb Ls x 13.50-lb. = 15.88″								
		1 Pl. 6″ x 7/16″ = 2.63								
		Total = 18.51″								
8	9	Light Sections.	10.98	2.46	10′-3″	50	35900		0.00	0.03 Down.
	39		11.01	"	"	"	35100	35700	0.06 S.	0.06 Up.
	78		11.03	"	"	"	36200		0.0072 S.	0.0086 Up.
	16		11.09	"	17′-5 1/8″	85	31400		0.03 S.	0.11 Up.
	81		11.05	"	"	"	32900	32800	0.0324 N.	0.0653 Up.
	82	4-4″ 7.5 x 1/4″ = 9.64″	11.11	"	"	"	34000		0.0117 S.	0.1597 Up.
		1 Pl. 7″ x 1/4″ = 1.75								
	27	Total = 11.39″	11.06	"	24′-7 3/16″	120	29100		0.07 N.	0.06 Down.
	46		11.11	"	"	"	29400	29700	0.0417	0.0194 Up.
	50		11.16	"	"	"	30400		0.0088 N.	0.1281 Down.
8A	100	Heavy Sections.	25.78	2.51	10′-5 1/2″	50	33000		0.0235 N.	0.0431 Down.
	126		25.86	"	"	"	33500	32900	0.0313 N.	0.0940 Down.
	127		25.83	"	"	"	32200		0.0261 N.	0.2400 Down.
	156		25.56	"	17′-9 3/8″	85	31300		0.0835 S.	0.2297 Up.
	160		25.57	"	"	"	25300	31400†	0.0010 S.	0.3393 Up.
	161	4-4″ 7.5 x 5/8″ = 22.20″	25.43	"	"	"	31500		0.0459 N.	0.1462 Up.
		1 Pl. 7″ x 5/8″ = 4.38								
	162	Total = 26.58″	25.66	"	25′-1 3/16″	120	27500		0.1019 S.	0.3075 Up.
	163		25.70	"	"	"	27000	27300	0.0848 N.	0.2025 Down.
	164		25.68	"	"	"	27500		0.0032 N.	0.760 Down.

* Tests have not been made. † Test no. 160 omitted in determining the average

TABLE 6.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES I & IA AND CARNEGIE SECTIONS.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	178	Light Sections.	11.93	2.24	3′-8 1/16″	20	38400		0.0261 S.	0.0261 Up.
	179		11.81	〃	〃	〃	38200	38200	0.0040 N.	0.0580 Down.
	180		11.79	〃	3′-8 3/4	〃	38000		0.0068 S.	0.0042 Down.
		4 L's 5″×3″×5/16″ = 9.60″								
		1 Pl. 6″×5/16″ = 1.875″								
		Total = 11.475″								
IA	175	Heavy Sections.	22.26	2.36	3′-11 1/16″	20	43200		0.0282 N.	0.0032 Down.
	176		22.33	〃	〃	〃	43300	43600	0.0578 N.	0.1023 Down.
	177		22.33	〃	〃	〃	44300		0.0385 N.	0.0462 Up.
		4 L's 5″×3″×5/8″ = 18.44″								
		1 Pl. 6″×5/8″ = 3.75								
		Total = 22.19″								
		Light Sections.	7.00	1.45	6′-0 7/16″	50	31500			
			7.00	〃	6′-0 1/2″	〃	30000	31000		
			7.00	〃	〃	〃	31400			
			7.00	1.45	10′-3 1/4″	85	30600			
			7.00	〃	10′-3 7/16″	〃	30000	30400		
		Carnegie 6″ H Col. x 23.8-lb.	7.00	〃	10′-3 1/4″	〃	30500			
		= 7.00″								
			7.00	1.45	14′-6″	120	26400			
			7.00	〃	14′-6 3/16″	〃	29300	27200		
			7.00	〃	14′-6″	〃	26000			
		Light Sections.	10.00	1.87	7′-9 1/2″	50	32800			
			10.00	〃	7′-9 3/8″	〃	33900	33500		
			10.00	〃	〃	〃	33800			
			10.00	1.87	13′-3″	85	31800			
			10.00	〃	〃	〃	31000	31700		
		Carnegie 8″ H Col. x 34.0-lb.	10.00	〃	〃	〃	32300			
		= 10.00″								
			10.00	1.87	18′-8 1/2″	120	30000			
			10.00	〃	〃	〃	28900	29200		
			10.00	〃	〃	〃	28700			

Areas given for Carnegie Sections are nominal.

TABLE 7.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 5, 5A & 5B.

Type.	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
		LIGHT AND HEAVY SECTIONS.								
5	107	Light Sections.	9.62	1.98	8'-3"	50	38000		0.0280 N.	0.0115 Down.
	122		9.65	"	"	"	38000	38000	0.0836 N.	0.0287 Down.
	123	A ⊢ A	9.65	"	"	"	38000		0.0052 S.	0.0209 Down.
	114		8.61	"	14'-0 5/16"	85	36000		0.1510 S.	0.0078 Down.
	116		9.66	"	"	"	34000	34300	0.1148 S.	0.0085 Up.
	119	Beth. 8" H Col. x 32-lb. = 9.17ª"	9.59	"	"	"	33000		0.1201 S.	0.0376 Up.
	110		8.64	"	19'-9 5/8"	120	33900		0.1148 N.	0.0084 Up.
	111		8.68	"	"	"	31000	32000	0.2140 N.	0.0136 Down.
	108		8.61	"	"	"	31000		0.1806 S.	0.0110 Up.
5A	106	Heavy Sections.	17.86	2.09	8'-8 1/2"	50	34000		0.1201 N.	0.0084 Up.
	120		17.82	"	"	"	36100	35400	0.0606 N.	0.0016 Up.
	121	A ⊢ A	17.86	"	"	"	36000		0.0284 N.	0.0033 Up.
	115		18.05	"	14'-9 5/8"	85	33900		0.0521 N.	0.0151 Down.
	117		17.73	"	"	"	32000	32300	0.0929 N.	0.0054 Down.
	118	Beth 8" H Col. x 62-lb. = 18.27ª"	17.77	"	"	"	31000		0.0470 S.	0.0277 Up.
	109		17.76	"	20'-10 13/16"	120	30000		0.1498 N.	0.0037 Up.
	112		17.97	"	"	"	30000	30000	0.2401 S.	0.0219 Down.
	113		17.73	"	"	"	30000		0.1610 N.	0.0221 Down.
		EXTRA HEAVY SECTIONS.								
5B	225	Extra Heavy Sections.	27.49	2.17	9'-0 1/2"	50	24000		0.0825 S.	0.0104 Down.
	226		27.18	"	"	"	24600	25200	0.1044 S.	0.0261 Down.
	182	A ⊢ A	27.50	"	"	"	27100		0.0522 N.	0.0033 Up.
	227		27.28	"	15'-4 7/16"	85	22600		0.2297 N.	0.0438 Down.
	228		27.27	"	"	"	23000	23500	0.0731 S.	0.0078 Up.
	183	Beth 8" H Col. x 91-lb. = 26.64ª"	27.45	"	"	"	24800		0.0553 S.	0.0016 Down.
	223		27.50	"	21'-8 3/8"	120	21000		0.3341 S.	0.0113 Up.
	224		27.54	"	"	"	21000	21300	0.2610 S.	0.0459 Up.
	181		27.57	"	"	"	21800		0.1903 S.	0.0031 Up.

TABLE 8.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES I, IA & IB.

Type.	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
		LIGHT AND HEAVY SECTIONS.								
	8	Light Sections.	11.13	2.24	9'-4"	50	32400		0.0350 S.	0.0000
	38	A	11.17	"	"	"	32800	32700	0.0700 S.	0.1000 Up.
	77		11.13	"	"	"	33000		0.0500 N.	0.0313 Up.
	18	A	11.49	"	15'-10 7/16"	85	32100		0.1200 S.	0.0500 Up.
	84		11.49	"	"	"	31400	31200	0.0252 S.	0.1281 Down.
	85	4 L's 5"x3"x 5/16" = 9.60"	11.35	"	"	"	30000		0.0626 N.	0.1368 Up.
		1 Pl. 6"x 5/16" = 1.875								
	30	Total = 11.475"	11.20	"	22'-4 13/16"	120	28600		0.0000	0.0600 Down.
	56		11.14	"	"	"	27600	28300	0.1175 S.	0.1091 Up.
	69		11.30	"	"	"	28700		0.0135 N.	0.1785 Down.
IA	93	Heavy Sections.	22.25	2.36	9'-10"	50	29000		0.0080 S.	0.0574 Down.
	96		22.13	"	"	"	28900	29200	0.0731 N.	0.0457 Up.
	97	A A	22.28	"	"	"	29700		0.0653 N.	0.0002 Up.
	91		22.134	"	16'-8 5/8"	85	28000		0.0475 S.	0.0883 Up.
	94		21.92	"	"	"	28200	28100	0.4053 N.	0.2636 Up.
	95	4 L's 5"x3"x 5/8" = 18.44"	22.05	"	"	"	28000		0.0764 N.	0.0825 Down.
		1 Pl. 6"x 5/8" = 3.75								
	92	Total = 22.19"	22.15	"	23'-7 3/16"	120	25000		0.1430 S.	0.2923 Down.
	98		22.10	"	"	"	25500	25400	0.3863 N.	0.0762 Up.
	99		22.10	"	"	"	25800		0.1326 N.	0.0606 Down.
		EXTRA HEAVY SECTIONS.								
		Extra Heavy Sections								
		A A								
IB	229		28.43	2.29	16'-2 5/8"	85	28200		0.0668 N.	0.0167 Down.
	230		28.37	"	"	"	27000	27700	0.0900 S.	0.0475 Down.
	231	4 L's 5"x3"x 13/16" = 23.36"	28.53	"	"	"	28000		0.0691 N.	0.0007 Up.
		1 Pl. 6"x 7/8" = 5.25								
		Total = 28.61"								

TABLE 9.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 1&1A–2&2A–4&4A.

Slenderness Ratio $\frac{l}{r}$=155.

Type	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure	
		Light Sections.								
		A								
	207		11.84	2.24	$28^{l}-11\frac{3}{16}^{u}$	155	27000		0.4176 S.	0.2506 Up.
1	218	A	11.69	"	"	"	25600	26200	0.1629 S.	0.6277 Down.
	222	$4 L 5''\times3''\times\frac{5}{16}''=9.60^{a}$	11.68	"	"	"	26000		0.1201 N.	0.3289 Down.
		$1 Pl.\ 6''\times\frac{5}{16}''=1.875$								
		Total $=11.475^{a}$								
		Heavy Sections.								
		A A								
	205		22.35	2.36	$30^{l}-5\frac{13}{16}^{u}$	155	22200		0.1629 S.	0.3967 Up
1A	206		22.35	"	"	"	23200	22700	0.0992 S.	0.4594 Down.
	208	$4 L 5''\times3''\times\frac{5}{8}''=18.44^{a}$	22.31	"	"	"	22700		0.2514 S.	0.4802 Down.
		$1 Pl.\ 6''\times\frac{5}{8}''=3.75$								
		Total $=22.19^{a}$								
		Light Sections.								
		A								
	215	$3\frac{1}{2}''$	10.39	2.31	$29^{l}-10\frac{1}{16}^{u}$	155	26100		0.8352 N.	0.1284 Up.
2	217	A	10.42	"	"	"	27800	26500	0.5116 N.	0.0668 Down.
	221	$2-6''$ Ex10.5-lb.$=6.18^{a}$	10.46	"	"	"	25600		0.9271 S.	0.0418 Down.
		$2 Pl^{s}\ 8''\times\frac{1}{4}''=4.00$								
		Total $=10.18^{a}$								
		Heavy Sections.								
		A								
	214	$3\frac{1}{2}''$	16.75	2.33	$30^{l}-1\frac{1}{8}^{u}$	155	24700		0.4959 S.	0.2589 Down.
2A	216	A	16.87	"	"	"	25000	24900	0.1958 N.	0.6160 Up.
	219	$2-6''$ Ex15.5-lb.$=9.12^{a}$	16.81	"	"	"	25000		0.0428 S.	0.2714 Down.
		$2 Pl^{s}\ 8''\times\frac{1}{2}''=8.00$								
		Total $=17.12^{a}$								
		Light Sections.								
		A								
	211		11.81	2.38	$30^{l}-8\frac{7}{8}^{u}$	155	24000		0.5324 N.	0.0002 Down.
4	212	A	11.93	"	"	"	23200	23600	0.7465 N.	0.0491 Down.
	220	$2-8''$ Ex11.25-lb.$=6.70^{a}$	11.84	"	"	"	23500		0.5116 S.	0.0040 Down.
		$1-8''$ I x18.00-lb.$=5.33$								
		Total $=12.03^{a}$								
		Heavy Sections								
		A								
	209		16.74	2.32	$29^{l}-11\frac{5}{8}^{u}$	155	24000		0.1754 S.	0.0569 Up.
4A	210	A	16.67	"	"	"	23000	23300	0.2453 S.	0.0512 Up.
	213	$2-8''$ I x18.75-lb.$=11.02^{a}$	16.33	"	"	"	23000		0.1973 S.	0.0063 Up.
		$1-8''$ I x20.50-lb.$=6.03$								
		Total $=17.05^{a}$								

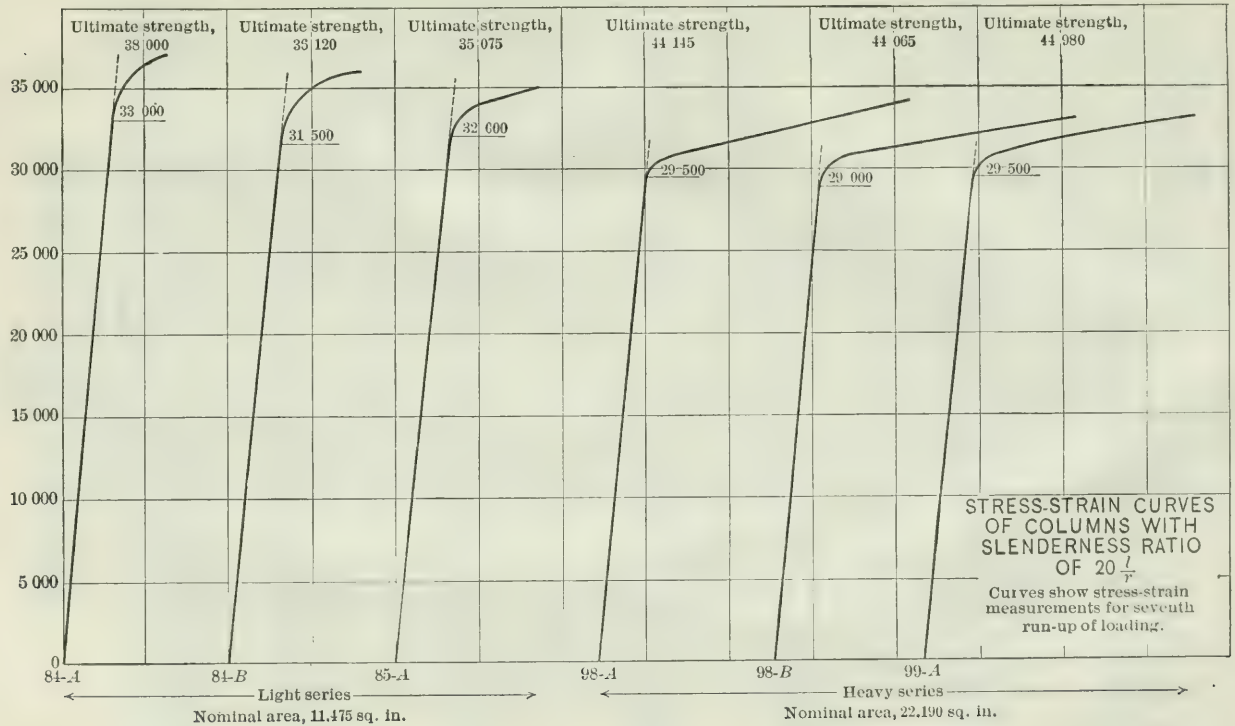

Ultimate strength, 38 000
Ultimate strength, 35 120
Ultimate strength, 35 075
Ultimate strength, 44 145
Ultimate strength, 44 065
Ultimate strength, 44 980
35 000
30 000
25 000
20 000
15 000
10 000
5 000
0
33 000
31 500
32 000
29 500
29 000
29 500
STRESS-STRAIN CURVES OF COLUMNS WITH SLENDERNESS RATIO OF 20 $\frac{l}{r}$
Curves show stress-strain measurements for seventh run-up of loading.
84-A
84-B
85-A
98-A
98-B
99-A
Light series
Nominal area, 11.475 sq. in.
Heavy series
Nominal area, 22.190 sq. in.

FIG. 5.

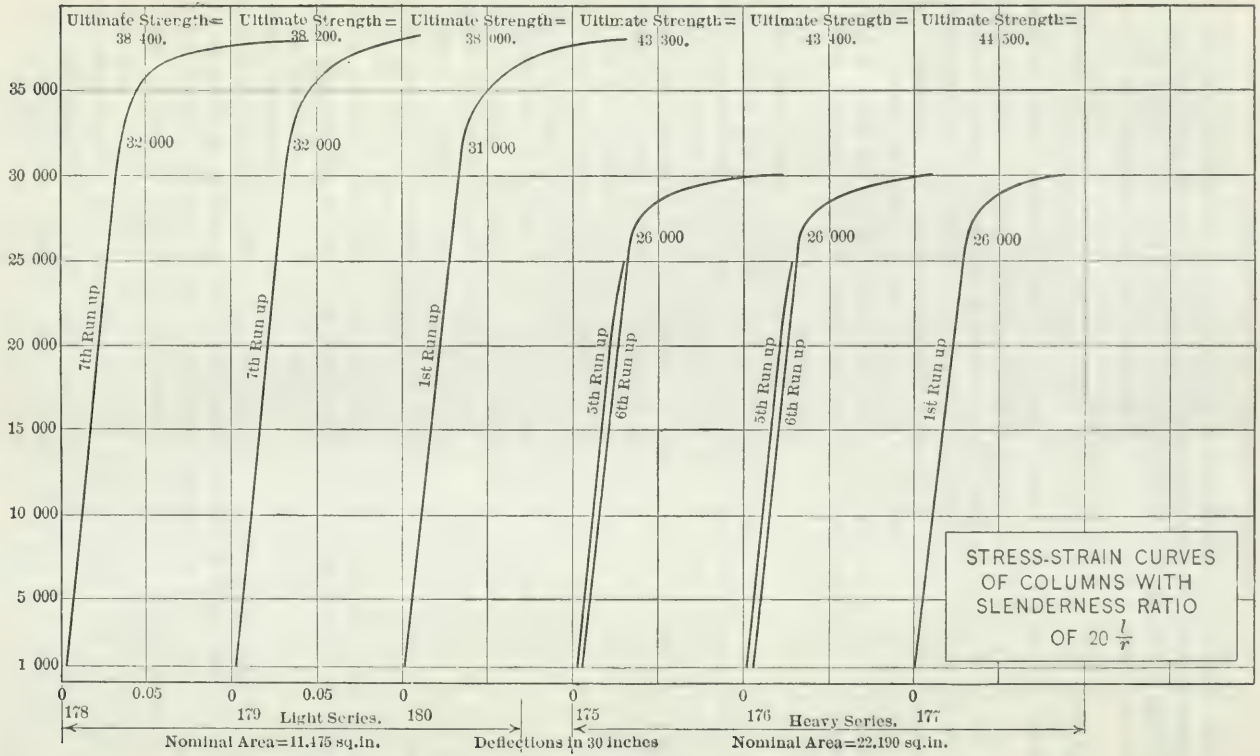

Ultimate Strength= 38 400.
Ultimate Strength= 38 200.
Ultimate Strength= 38 000.
Ultimate Strength= 43 300.
Ultimate Strength= 43 400.
Ultimate Strength= 44 500.
35 000
30 000
25 000
20 000
15 000
10 000
5 000
1 000
32 000
32 000
31 000
26 000
26 000
26 000
7th Run up
7th Run up
1st Run up
5th Run up
6th Run up
5th Run up
6th Run up
1st Run up
STRESS-STRAIN CURVES OF COLUMNS WITH SLENDERNESS RATIO OF 20 l/r
0
0.05
0
0.05
0
0
0
0
0
178
179
Light Series.
180
175
176
Heavy Series.
177
Nominal Area=11.175 sq.in.
Deflections in 30 inches
Nominal Area=22.190 sq.in.

FIG. 6.

A question arose as to the tendency of the material to flow under load, and, in order to secure information on this point, a special time test was inaugurated. This is given as No. 54, where it will be seen that various loadings were applied and then held while the compressometers were read at intervals of 5 min. until the flow of the metal ceased. Although the results are interesting, and might be a matter of speculation in beginning a new programme, the variation in the failure point from similar columns does not seem to show enough difference to warrant any extensive carrying out of time tests in the present programme.

As stated in the foregoing report, the programme was arranged "with the intention, as far as practicable, of eliminating all variables except that of form, and was subdivided so as to consider the effect of light and heavy sections." The Committee felt at that time that it was better to confine itself to a rather narrow programme, with the use of columns having a cross-section arranged so as to avoid lacing or battens, rather than to attempt to cover the effects of details and unsymmetrical sections. The Committee was influenced somewhat in the choice of certain forms of columns by an opinion held by some engineers that columns failed largely through the crippling of the thin outstanding legs. For example, in a discussion published in 1908,* George E. Gifford, M. Am. Soc. C. E., expressed the opinion that angles $\frac{1}{2}$ in. thick would be more than twice as strong as angles $\frac{1}{4}$ in. thick; and the late T. H. Johnson, M. Am. Soc. C. E., suggested the use of bulb-angles, believing that the bulb on the outstanding leg gave stiffness to the columns. He called attention to the superior behavior of Z-bar columns, "evidently due to the support of the horizontal leg afforded by the outer leg of the Z".

On this account, the Committee selected a bulb-angle column, Type 7, and a Z-bar column, Type 8, for test, and designed a column, Type 6, made up of an I-beam with two plates riveted to the flanges. The Committee realized that this Type 6 column was not a satisfactory form, but wanted to study the effect of the outstanding edges of the flange plates. Tables 1 to 9 show the record of the column tests published in the Progress Reports of 1916 and 1917. These records call attention to the decrease in unit ultimate strength of the heavy sections when compared with the light ones. This, with the single exception of Type 6, slenderness ratio $\frac{l}{r} = 50$ (see Table 4) was true for all the different types of columns.

The results, as shown by the foregoing tests, though demonstrating the value of a thicker plate in the short columns of Type 6, hardly prove or disprove the original question before the Committee, as to

* *Proceedings*, Am. Soc. for Testing Materials.

the influence of the thin outstanding legs. It is possible that, for very thin metal, the ratio of the thickness of the outstanding leg to its width might have an effect on the strength of the column, but, for the thicknesses of the material which have been used in the programme of tests outlined by the Committee, it is evident that the ratio of thickness to width is not a controlling factor.

In seeking to account for the falling off in strength of the heavy material, the Committee learned that it was necessary to look beyond the differences in the ratios of widths to thicknesses of outstanding legs, and variations of cross-section. The only remaining element which could be charged with responsibility appeared to be the metal itself, and though the intention had been to secure material of a uniform grade, a more careful and thorough investigation disclosed the fact that the attempt had not been successful.

The physical characteristics and strength of structural steel are affected by the amount of working which it receives in passing through the rolls, and, for the same size of ingot, the heavy material does not receive as much working as the light; consequently, for the same chemical composition, the heavier material is weaker. In ordering the steel for the column tests, the Committee provided for this difference in working by allowing the mill to change the carbon content so as to endeavor to produce the same strength for the light material as for the heavy. When the attention of the Committee and the Bureau of Standards was drawn to the falling off in the unit strength of the heavy material, the Bureau of Standards investigated the record of the physical and chemical tests made at the mill when the steel was rolled. The complete record of these physical and chemical tests is shown in Table 10, and it will be seen that this record gives no indication of the difference in strength between the heavy and light material.

A note concerning the choice of the original specifications might be in order before commenting on the record of specimen tests.

At the time the Bureau of Standards offered to make the column tests, Dr. Stratton requested the Committee to write the specifications and secure prices. It was desired to have the material for the columns as nearly uniform as possible, the thought being to have as much of it as possible come from a single heat. The specifications (page 1591) are the result of the conference of the Committee with the steel manufacturers, and, as stated before, they are very much more rigid than the average structural specifications. The Committee accepted some material which did not fully meet the rigid specifications, but it was considered at that time to be fully equal to, or better than, the ordinary grade of structural steel, which has an allowable variation of 5 000 lb. per sq. in., more or less, from a desired ultimate strength of 60 000 lb. per sq. in.

TABLE 10.—(*Continued.*)

Heat No.	Type.	Lot.	Specimen cut from:	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
	Specifications.			37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
14414	2*a*	3	8 x ½	40 160*	61 360	32.5*	60.8*	0.21	0.44	0.010	0.038	½ Cup.
C11393	2*a*	3	6 Ch. 15½-lb.	35 430*	58 180*	30.0	52.6	0.24	0.44	0.010	0.041*	Angular.
47096	3	1	9½ x ¼	37 670	54 980*	29.2	57.4	0.17	0.44	0.014	0.029	½ Cup.
47096	3	1	9½ x ¼	36 550*	53 500*	30.6*	46.8*	0.17	0.44	0.014	0.029	½ Cup.
46201	3	1	5-in. Ch. 6½	42 020*	59 240	23.2*	67.2*	0.20	0.48	0.020	0.035	½ Cup.
46201	3	1	do	44 700*	55 900*	26.3	57.7	0.20	0.48	0.020	0.035	Cup.
47096	3	1	8 x ¼	38 300	54 250*	25.0*	48.8*	0.17	0.44	0.014	0.029	Angular.
38054	3*a*	2	5-in. x 11.5-lb. Ch.	36 790*	58 100*	34.7*	59.6	0.20	0.40	0.020	0.032	Angular.
41029	3*a*	2	9½ x ½	39 000	60 350	31.2*	54.7	0.22	0.44	0.027	0.036	Angular.
44200	4	1	8-in. Ch. 11¼	40 800*	58 600	27.8	48.7*	0.20	0.45	0.018	0.028	Irregular.
44200	4	1	do	38 650	60 700	25.0*	56.2	0.20	0.45	0.018	0.028	½ Cup.
27225	4	1	8-in. I 18-lb.	34 960*	58 280*	28.5	58.2	....	....	.....	.....	Angular.
27225	4	1	do	32 100*	54 600*	30.0	56.4	....	....	.....	.....	½ Cup.
C4289	4	3	8-in. Ch. 11¼-lb.	36 260*	58 240*	26.2	57.9	0.21	0.47	0.012	0.039	Irregular.
10184	2*a*	2	8 x ½ Pl.	37 430	60 460	27.5	51.1*	0.22	0.50	0.011	0.040	Angular.
10184	2*a*	2	do	37 420	61 620*	27.5	52.2	0.22	0.50	0.011	0.040	Angular.
48332	4*a*	2	8-in. x 18¾-lb. Ch.	38 590	59 240	30.5*	49.6*	....	....	.....	.....	½ Cup.
48332	4*a*	2	do	38 020	58 880	30.5*	49.8*	....	....	.....	.....	Cup.
C5315	4*a*	3	8-in. I 20.5-lb.	36 990*	59 700	30.0	57.9	0.23	0.47	0.011	0.037	Angular.
42239	4*a*	2 or 3	8-in. 20½-lb. I	37 990	59 740	28.7	54.0	0.21	0.50	0.010	0.036	do.
42239	4*a*	2 or 3	do	36 370*	59 600	27.7	53.6	0.21	0.50	0.010	0.036	do.
47096	6	1	11 x 5/16	35 140*	54 740*	30.7*	55.9	0.17	0.44	0.014	0.029	Angular.
47096	6	1	11 x 5/16	32 550*	53 200*	29.4	54.9	0.17	0.44	0.014	0.029	Angular.
42171	6	1	11 x 5/16	37 570	58 100*	30.5*	60.1*	0.20	0.44	0.010	0.028	½ Cup.
42171	6	1	11 x 5/16	32 990	57 830*	31.4*	54.5	0.20	0.44	0.010	0.028	Angular.
39235	6	1	10 I 25-lb.	38 580	63 600*	28.7	55.0	....	....	.....	.....	Angular.
39235	6	1	do	35 370*	62 590*	26.2	58.2	....	....	.....	.....	½ Cup.
39235	6	1	do	32 300*	59 800*	27.0	52.8	....	....	.....	.....	Angular.
37261	6	1	do	37 680	55 400*	30.0	61.6*	....	....	.....	.....	Cup.
37261	6	1	do	29 100*	53 200*	32.0*	54.4	....	....	.....	.....	Angular.
41029	6*a*	2	11 x 5/8	39 090*	60 540	28.0	53.9	0.22	0.44	0.027	0.036	Cup.
42239	6*a*	2	10-in. — 35-lb.	37 930	60 300	28.0	47.2*	0.21	0.50	0.010	0.036	Irregular.
42239	6*a*	2	do	37 930	60 250	26.2	46.8*	0.21	0.50	0.010	0.036	Angular.

* Not within limits of specifications.

TABLE 10.—Record of Physical and Chemical Specimen Tests Made by Mill at Pittsburgh.

Heat No.	Type.	Lot.	Specimen cut from:	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
	Specifications.			37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
11105	1 & 7	1	6 x $\frac{5}{16}$	36 950*	60 860	27.5	57.0	0.20	0.47	0.013	0.038	Angular.
11105	1 & 7	1	6 x $\frac{5}{16}$	35 180*	59 680	25.5*	57.3	0.20	0.47	0.013	0.038	do.
11105	1 & 7	1	6 x $\frac{5}{16}$	34 450*	60 260	28.5	50.5*	0.20	0.47	0.013	0.038	do.
42171	1	1	5 x 3 x $\frac{5}{16}$∠	34 270*	57 990*	28.7	55.1	0.20	0.44	0.010	0.028	½ Cup.
42171	1	1	5 x 3 x $\frac{5}{16}$∠	34 070*	57 740*	28.0	55.0	0.20	0.44	0.010	0.028	Angular.
42171	1	1	5 x 3 x $\frac{5}{16}$	29 700*	56 000*	28.0	51.4*	0.20	0.44	0.010	0.028	Angular.
11105	1 & 7	1	6 x $\frac{5}{16}$	32 950*	59 730	29.5	54.8	0.20	0.47	0.013	0.038	do.
C13333	1	3	5 x 3 x $\frac{5}{16}$∠	36 250*	61 600*	27.5	51.8*	0.22	0.47	0.037*	0.047*	Angular.
C13333	1	3	5 x 3 x $\frac{5}{16}$	36 240*	58 050*	27.5	50.7*	0.22	0.47	0.037*	0.047*	Cup.
C14304	1	3	6 x $\frac{5}{16}$	37 040	59 369	27.5	56.0	0.25	0.50	0.012	0.040	Angular.
C1267	1	3	5 x 3 x $\frac{5}{16}$	36 350*	60 860	30.0	60.9*	0.21	0.42	0.016	0.040	Cup.
C13354	1*a*	2 or 3	5 x 3 x $\frac{5}{8}$	38 800	60 920	27.5	58.2	0.21	0.50	0.010	0.032	½ Cup.
C13354	1*a*	2 or 3	5 x 3 x $\frac{5}{8}$	38 280	61 430	30.0	58.4	0.21	0.50	0.010	0.032	½ Cup.
38054	1*a*	2	6 x $\frac{5}{8}$	36 940*	58 400*	35.5*	55.6	0.20	0.40	0.020	0.032	Angular.
1257	1*a*	2	5 x 3 x $\frac{5}{8}$	38 300	60 460	28.7	44.2*	0.22	0.50	0.015	0.030	Angular.
1257	1*a*	2	5 x 3 x $\frac{5}{8}$	39 100*	58 970	27.5	49.3*	0.22	0.50	0.015	0.030	Angular.
10032	1*a*	2	5 x 3 x $\frac{5}{8}$	38 280	57 890*	30.0	53.8	0.20	0.42	0.012	0.039	Cup.
10032	1*a*	2	5 x 3 x $\frac{5}{8}$	38 240	58 760	30.0	59.3	0.20	0.42	0.012	0.039	Cup.
C1267	1*a*	3	5 x 3 x $\frac{5}{8}$	36 600*	59 380	28.7	53.4	0.21	0.42	0.016	0.040	Angular.
C10133	1*a*	3	6 x $\frac{5}{8}$	37 100	61 460	27.5	55.6	0.20	0.46	0.040*	0.037	Angular.
C5400	1*b*	3	6 x $\frac{7}{8}$	36 880*	59 520	27.5	47.3*	0.21	0.47	0.022	0.032	Cup.
C1267	1*b*	3	5 x 3 x $\frac{13}{16}$	35 910*	59 580	28.7	56.3	0.21	0.42	0.016	0.040	Angular.
C1267	1*b*	3	5 x 3 x $\frac{13}{16}$	36 260*	59 740	28.7	58.9	0.21	0.42	0.016	0.040	Angular.
C1267	1*b*	3	5 x 3 x $\frac{13}{16}$	36 650	59 480	30.0	59.7	0.21	0.42	0.016	0.040	Cup.
31082	2	1	6 x 10½-lb. Ch.	35 270*	57 630*	30.2*	61.3*	....	....	.....	.....	½ Cup.
31082	2	1	do	36 500	59 700	28.6	57.7	....	....	.....	.....	Cup.
47096	2	1	8 x ¼	38 670	55 520*	31.2*	54.5	0.17	0.44	0.014	0.029	½ Cup.
42171	2	1	8 x ¼	37 840	58 130*	30.0	58.5	0.20	0.44	0.010	0.028	Angular.
42171	2	1	8 x ¼	34 310*	57 550*	30.0	49.8*	0.20	0.44	0.010	0.028	Angular.
14414	2	3	8 x ¼	40 510*	63 660*	27.5	61.1*	0.21	0.44	0.010	0.038	Cup.
C11393	2	3	6 Ch 10½-lb.	35 900*	61 700*	27.5	54.5	0.24	0.44	0.010	0.040	Angular.
38054	2*a*	2	6 15½-lb. Ch.	36 500*	56 990*	30.2*	51.7*	0.20	0.40	0.020	0.032	Angular.
38054	2*a*	2	6 15½-lb. Ch.	36 220*	59 840	27.5	47.3*	0.20	0.40	0.020	0.032	Cup.

*Not within limits of specifications.

TABLE 10 (*Continued*)—REJECTED MATERIAL.

Heat No.	Type.	Lot.	Specimen cut from:	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
	Specifications.			37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	Variable	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
31215	.....	.....	5 x 3 x 5/8	36 540*	54 860*	32.5*	60.0	0.21	0.48	0.016	0.032	½ Cup.
31215	.....	.....	do.	36 470*	54 570*	31.2*	58.7	0.21	0.48	0.016	0.032	½ Cup.
31215	.....	.....	do.	36 010*	54 060*	32.5*	60.1*	0.21	0.48	0.016	0.032	½ Cup.
34428	.....	.....	8 lb. x 18¾ lb. Ch.	37 550	55 340*	29.7	52.7	0.20	0.49	0.019	0.033	Angular.
34228	.....	.....	do.	36 650*	55 320*	30.0	52.3	0.20	0.49	0.019	0.033	do.
44292	.....	.....	do.	36 290*	56 420*	31.0*	52.7	0.20	0.40	0.010	0.031	Irregular.
44292	.....	.....	do.	38 560	55 400*	30.7*	48.8*	0.20	0.40	0.010	0.031	Angular.
39208	.....	.....	8 in. I 18		64 320*	26.0	49.5*	....	....	.....	.....	Irregular.
39208	.....	.....	8 in. I 15		63 580*	26.2	51.8*	....	....	.....	.....	½ Cup.
39208	.....	.....	8 in. I 18	34 100*	61 300	27.0	51.5*	....	....	.....	.....	Angular.
47250	.....	.....	10 in. I 25	36 360*	56 450*	32.2*	59.5	....	....	.....	.....	Cup.
47250	.....	.....	10 in. I 25	31 200*	54 600*	30.0	58.2	....	....	.....	.. ..	Cup.

* Not within limits of specifications.

TABLE 10.—(*Continued.*)

Heat No.	Type.	Lot.	Specimen cut from:	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
	Specifications.			37 000 to 39 000	58 500 to 61 500	20 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
42171	??	1	5 in. bulb 10.1-lb.	33 100*	56 300*	28.0	50.8*	0.20	0.44	0.010	0.028	Angular.
47096	8	1	7 x ¼ Pl.	39 860*	55 490*	31.5*	50.8*	0.17	0.44	0.014	0.029	Angular.
47096	8	1	7 x ¼	37 800	53 500*	30.6*	54.3	0.17	0.44	0.014	0.029	Cup.
42171	8	1	4-in. Z 8½-lb.	39 650*	57 640*	31.2*	56.1	0.20	0.44	0.010	0.028	Angular.
42171	8	1	do	39 430*	58 340*	31.2*	59.5	0.20	0.44	0.010	0.028	Irregular.
42171	8	1	do	39 050*	56 700*	32.5*	55.3	0.20	0.44	0.010	0.028	Angular.
42171	8	1	do	39 370*	56 100*	33.0*	55.4	0.20	0.44	0.010	0.028	Irregular.
7463	8a	2	7 x ⅝ Pl.	35 750*	58 240*	27.5	51.1*	0.20	0.46	0.019	0.034	Angular.
7463	8a	2	do	38 740	59 620	28.7	56.0	0.20	0.46	0.019	0.034	Angular.
7463	8a	2	4-in. Z	38 260	58 240*	28.7	52.2	0.20	0.46	0.019	0.036	Angular.
11105	10	1	5¼ x ¼	39 220*	60 060	27.5	55.9	0.20	0.47	0.013	0.038	Angular.
11105	10	1	do	37 400	59 420	29.0	60.7*	0.20	0.47	0.013	0.038	½ Cup.
11105	10	1	do	33 950*	59 510	31.5*	55.8	0.20	0.47	0.013	0.038	Cup.
11105	10	1	do	35 000*	59 960	29.3	52.8	0.20	0.47	0.013	0.038	Angular.
47096	10	1	2 x 2 x ¼ ∠	36 620*	56 800*	31.2*	61.7*	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	36 600*	56 360*	27.5	60.2*	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	36 200*	55 500*	32.0*	59.7	0.17	0.44	0.014	0.029	Angular.
47096	10	1	do	35 700*	54 650*	30.0	58.8	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	9 x ¼	38 780	54 640*	31.2*	56.5	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	36 100*	52 800*	31.2*	46.0*	0.17	0.44	0.014	0.029	½ Cup
42171	10	1	2 x 2 x ¼	41 140*	61 140	27.5	60.7*	0.20	0.44	0.010	0.028	½ Cup.
42171	10	1	do	40 530*	60 940	27.5	59.8	0.20	0.44	0.010	0.028	Angular.
42171	10	1	do	41 110*	60 940	29.8	59.8	0.20	0.44	0.010	0.028	½ Cup.
42171	10	1	do	40 570*	59 860	27.5	62.4*	0.20	0.44	0.010	0.028	Cup.
42171	10	1	do	36 960*	58 930	27.0	57.7	0.20	0.44	0.010	0.028	Angular.
42171	10	1	do	36 450*	58 800	29.5	58.7	0.20	0.44	0.010	0.028	½ Cup.
7463	10a	2	5¼ x 7/16	37 010	62 320*	26.7	55.3	0.20	0.46	0.019	0.034	½ Cup.
4038	10a	2	2 x 2 x 7/16	38 280	61 260	27.5	56.9	0.20	0.47	0.014	0.039	Cup
41239	10a	2	9 x 7/16	37 580	61 450	28.7	52.1	0.22	0.44	0.027	0.036	½ Cup.

* Not within limits of specifications.

Not finding an explanation of the falling off in strength of the heavy material in the record of mill specimen tests, the Bureau of Standards then took some of the long columns, which had already been tested, and cut them into lengths to give a slenderness ratio of $\frac{l}{r} = 20$.

These short columns showed that the unit ultimate strength of the heavy columns was considerably more than that of the light ones. Both series indicated a fairly definite point at which permanent set occurred, and showed that this point was lower for the heavy columns than for the light ones, indicating that the increased ultimate strength of the heavy ones came about from the block action of the short, heavy material, and that the elastic limits or yield points are the true indicators of the strength of the two different thicknesses of material. See Fig. 5, from the 1916 Progress Report.

Tests of short columns having a slenderness ratio of $\frac{l}{r} = 20$, made of material which had not been previously stressed, confirmed these results and the belief that the strength of columns is governed, not by the ultimate tensile strength of the material, but by the point at which there is marked departure from an elastic condition. See Fig. 6, from the 1917 Progress Report.

To investigate the question of this critical point, the Bureau of Standards proposed some supplementary specimen tension and compression tests, to be made from pieces, 5 ft. in length, which had been provided from each melt at the time the material was rolled. These 5-ft. pieces were in the nature of coupon material from the columns, and had not been subjected to stresses in the testing of the columns.

Tables 11 to 15 show the record of the supplementary specimen tests made on the coupon material ordered from the same melts as the material for the full-size columns, and Table 22 is the summary of the averages of Table 10 and Tables 11 to 15. When we compare the results shown by Tables 11 to 15 with the mill specimen tensile tests made from these same melts, Table 10, and both with the table of full-size column tests, it will be seen that the ultimate tensile specimen strength is not a correct indicator of the strength of the columns. The ultimate strengths shown on the supplementary specimen tests are very close to the ultimate strengths given for the specimen mill tests, and neither the ultimate strengths from the supplementary tests nor from the mill tests indicate the falling off in strength of the thicker material. It is also evident that the yield point, as recorded by the ordinary commercial tensile specimen tests, even when the machine is run at comparatively slow speeds, as was done in the Pittsburgh Laboratory of the Bureau of Standards, does not give the correct index of the

TABLE 11.

ABSTRACT OF RECORD OF SPECIMEN TESTS.

MADE BY BUREAU OF STANDARDS.

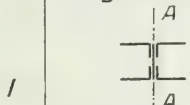
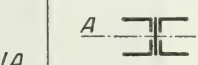
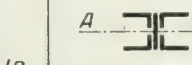
Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point.
I	Light Sections.  4 Ls 5″x3″x 5/16″ = 9.60□″ 1 Pl. 6″x 5/16″ = 1.875 Total = 11.475□″	II	5″ leg of 5″x3″x 5/16″ L.	57100	35100
		III	"	53900	33200
		II	Root of 5″x3″x 5/16″ L.	65800	33500
		III	"	64000	33200
		II	3″ leg of 5″x3″x 5/16″ L.	59000	32700
		21	6″x 5/16″ Pl.	58400	33000
		22	"	58600	33000
		112	"	54900	30800
		Compression Tests.			
		Specimen no	Specimen	Ur of Specimen	Useful Limit Point.
		B 18	5″x3″x 5/16″ L.	12.3	34800
		12	"	12.7	33400
		21	6″x 5/16″ Pl.	—	31000
		21	"	—	31000
		112	"	—	27300
IA	Heavy Sections.  4 Ls 5″x3″x 5/8″ = 18.44□″ 1 Pl. 6″x 5/8″ = 3.75 Total = 22.19□″	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point
		113	5″ leg of 5″x3″x 5/8″ L.	56700	28800
		60	"	55600	30800
		Compression Tests.			
		Specimen no	Specimen	Ur of Specimen	Useful Limit Point.
		1	5″x3″x 5/8″ L.	12.6	28600
		2	"	12.9	25000
		A	"	12.7	27300
IB	Extra Heavy Sections.  4 Ls 5″x3″x 13/16″ = 23.36□″ 1 Pl. 6″x 7/8″ = 5.25 Total = 28.61□″	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point.
		109	Leg of 5″x3″x 13/16″ L.	57000	31100

TABLE 12.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point
2	Light Sections. 2-6"[s 10.5-lb=6.18□" 2 Pl[s 8"x1/4"=4.00 Total = 10.18□"	27	8"x1/4" Pl.	57700	32700
		17	"	56000	29700
		116	"	59600	32200
		44	Root of 6"[10.5-lb.	59700	28400
		115	"	63800	31800
		37	Web of 6"[10.5-lb.	56700	35800
		44	"	56600	33000
		115(22A)	"	60500	37000
		Compression Tests.			
		Specimen no.	Specimen	℄ of Specimen	Useful Limit Point
		37	6"[10.5-lb	19.0	33300
		44	"	"	35000
		115	"	"	33900
		17	8"x1/4" Pl.	—	30400
		27	"	—	22800
		116	"	—	31500
2A.	Heavy Sections. 2-6"[s 15.5-lb=9.12□" 2 Pl[s 8"x1/2"=8.00 Total = 17.12□"	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point.
		118	8"x1/2" Pl.	58400	31000
		69	"	61500	35000
		66	Web of 6"[15.5-lb.	53500	23500
		117	"	58900	32100
		66	Root of 6"[15.5-lb.	57300	30000
		117	"	60900	29800

TABLE 13.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

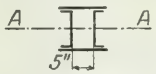
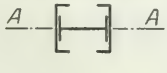
Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
3	Light Sections  2-5"[s 6.5-lb.=3.90□" 2 Pl. 9 1/2 x 1/4 =4.75 Total =8.65□"	38	Web of 5"[6.5-lb.	54200	36500
		43	"	55500	39700
		38	Root of 5"[6.5-lb.	55700	31000
		43	"	56400	29100
		Compression Tests.			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point
		38	5"[6.5-lb.	20.2	35200
4	Light Sections.  2-8"[s 11.25-lb.=6.70□" 1-8"I 18.0-lb.=5.33 Total =12.03□"	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point
		42.2	Web of 8"[11.25-lb.	57700	36000
		42.3	"	60000	40800
		48	"	57400	37900
		42	Root of 8"[11.25-lb.	59800	32900
		48	"	61600	33200
		119	"	60000	27600
		50	Flange of 8"I 18.0-lb.	56100	34500
		50	"	56300	35600
		50	Root of 8"I 18.0-lb.	58000	28900
		55	"	57600	31000
		120	"	58900	27200
		55	Web of 8"I 18.0-lb.	55000	36500
		55	"	56000	37800
		Compression Tests			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point
		119	8"[11.25-lb.	16.0	27500
		G	"	15.9	40000
		H	"	16.0	40700
		120	8"I 18.0-lb.	11.8	29000
		F	"	12.0	32000
		E	"	11.9	31600

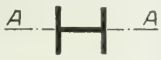
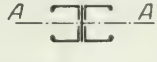
TABLE 14.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
4A	Heavy Sections. 2-8"[s 18.75-lb=11.02$^{a''}$ 1-8"I 20.50-lb=6.03 Total = 17.05$^{a''}$	80	Web of 8"[18.75-lb	55600	23500
		483-32	"	60200	23700
		79	Root of 8"[18.75-lb	59900	27000
		80	"	61200	23000
		121	"	60100	30200
		483-32	"	63400	29200
		81 W	Web of 8"I 20.5-lb.	58000	30000
		82	"	57300	28000
		83	"	57700	27500
		122	"	56100	31300
		81	Root of 8"I 20.5-lb.	62700	26600
		122	"	61200	25100
		83	"	62500	27300
		Compression Tests.			
		Specimen no.	Specimen	L/r of Specimen	Useful Limit Point
		79	8"[18.75-lb.	16.8	26000
		80	"	16.6	26000
		I	"	16.7	28500
		121	"	16.6	34000
		83	8"I 20.5-lb	12.0	28700
		122	"	12.3	29200
		B	"	12.2	25700
		D	"	12.3	30000

TABLE 15.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
5	Light Sections. 8"H Col. 32.0-lb. = 9.17ª"	19BX	Flange of 8"H Col. 32.0-lb.	58900	25600
		19BXC	Web of 8"H Col. 32-lb.	58900	37000
		19BX	Root of 8"H Col. 32-lb.	64000	38000
5A	Heavy Sections. 8"H Col. 62.0-lb. = 18.27ª"	15CX(C)	Web of 8"H Col. 62.0-lb.	57500	32500
		15CX	Flange of 8"H Col. 62.0-lb.	58100	33800
		15CX(M)	Root of 8"H Col. 62.0-lb.	58000	33700
5B	Extra Heavy Sections.  8"H Col. 91.0-lb. = 26.64ª"	24AXC	Web of 8"H Col. 91.0-lb.	56500	25000
		24BXC	"	56500	23100
		24CXC	"	57500	26600
		24AXM	Middle of Flange of 8"H Col. 91.0-lb.	56100	18900
		24CXM	"	56300	26400
		24CX	"	56300	26000
		24BXM	"	56400	20900
		24AX	End of Flange of 8"H Col. 91.0-lb.	56400	17600
		24CX	"	55100	23300
		24AX	"	56700	18500
		24BX	"	56700	21000
		24BX	"	56800	24500
7	Light Sections.  4-5"Bulb Ls 10.1-lb. = 11.88ª" 1 Pl. 6" x 5/16" = 1.88 Total = 13.76ª"	2	Root of 5"Bulb L. 10.1-lb.	61600	33900
		6	Bulb of 5"Bulb L. 10.1-lb.	61700	33500
		9	"	61500	33000

Compression Tests.			
Specimen no.	Specimen	Yr of Specimen	Useful Limit Point
—	5"Bulb L. 10.1-lb.	17.6	34000
6	"	17.8	34000
9	"	17.7	35000

strength of the material. It appears necessary, therefore, in order to predict the strength of a column, to determine the nature of the metal by some other means than those generally used.

In the Progress Report of 1917, the Committee mentioned the discussion held by the American Society for Testing Materials at its Annual Meeting, June, 1916, on the relation between proportional limit, elastic limit, and yield point. For the purpose of studying the column tests, the Committee gave careful consideration to this discussion to find whether it was possible to determine some point which, for practical purposes, might be easily located, clearly defined, and at the same time represent the limit where the metal ceases to have structural value. None of the terms defined by the discussions of the American Society for Testing Materials has appealed to the Committee as having these qualities. In searching for a more satisfactory definition, the Committee considered a modification of the suggestion made some years ago by the late J. B. Johnson, M. Am. Soc. C. E. The Committee has defined the critical point as the point which is determined graphically by drawing a line tangent to the envelope of the stress-strain curve, having a slope of one-half that of the last run-up line for its straight, or nearly straight, portion.

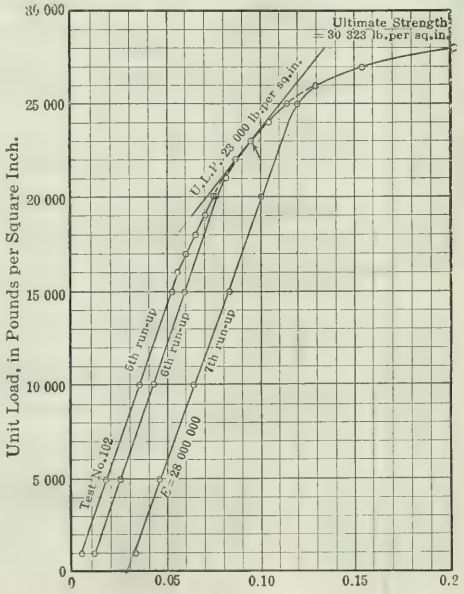


FIG. 7.

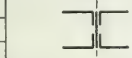
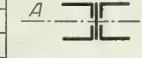
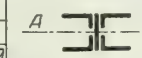
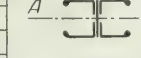
Fig. 7 illustrates this method of determination. So as not to confuse this with former definitions of yield point or elastic limit, the Committee has adopted a new term, and calls this the Useful Limit Point, or U. L. P.

In straining a column, there is a point beyond which its structural value is uncertain, and consequently unsafe to rely upon. This point lies somewhere above the region of perfect elasticity and well below the place where manifest yielding occurs. For the study of column tests, the U. L. P. as above defined seems to fulfill these conditions satisfactorily. Careful observations and plotting of the stress-strain curve locate it without chance for controversy. It will be noted that the method is applicable to both tension and compression tests.

TABLE 16.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES I, IA, IB & 7.

Type	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
I	8	Light Sections.	11.13	2.24	9'-4"	50	32400		28300	
	38	A	11.17	"	"	"	32800	32700	29200	28800
	77		11.13	"	"	"	33000		29000	
	18	A	11.49	"	15'-10 7/16"	85	32100		26000	
	84		11.49	"	"	"	31400	31200	30000	27800
	85	4 L's 5" x 3" x 5/16" = 9.60a"	11.35	"	"	"	30000		27500	
		1 Pl. 6" x 5/16" = 1.875								
	30	Total = 11.475a"	11.20	"	22'-4 13/16"	120	28600		25400	
	56		11.14	"	"	"	27600	28300	26300	26600
	69		11.30	"	"	"	28700		28000	
IA	93	Heavy Sections.	22.25	2.36	9'-10"	50	29000		25000	
	96		22.13	"	"	"	28900	29200	24000	24700
	97		22.28	"	"	"	29700		25000	
	91		22.13	"	16'-8 5/8"	85	28000		25700	
	94		21.92	"	"	"	28200	28100	24700	25300
	95	4 L's 5" x 3" x 5/8" = 18.44a"	22.05	"	"	"	28000		25500	
		1 Pl. 6" x 5/8" = 3.75								
	92	Total = 22.19a"	22.15	"	23'-7 3/16"	120	25000		23000	
	98		22.10	"	"	"	25500	25400	23500	23300
	99		22.10	"	"	"	25800		23300	
IB		Extra Heavy Sections.								
										
	229		28.43	2.29	16'-2 5/8"	85	28200		26500	
	230		28.37	"	"	"	27000	27700	26000	26200
	231	4 L's 5" x 3" x 13/16" = 23.36a"	28.53	"	"	"	28000		26000	
		1 Pl. 6" x 7/8" = 5.25								
		Total = 28.61a"								
7*	5	Light Sections.	13.33	2.48	10'-4"	50	34300		30300	
	37		13.30	"	"	"	32700	33400	30600	30600
	76		13.33	"	"	"	33200		31000	
	15		13.40	"	17'-6 13/16"	85	34000		30800	
	89		13.43	"	"	"	30000	31600	29000	29400
	90	4-5" Bulb L's x 10.1 lb. = 11.88a"	13.29	"	"	"	30700		28500	
		1 Pl. 6" x 5/16" = 1.88								
	28	Total = 13.76a"	13.36	"	24'-9 5/8"	120	28300		28400	
	48		13.34	"	"	"	26900	28100	25900	26600
	51		13.33	"	"	"	29000		25600	

* Shapes for Type 7A (Heavy Sections) cannot be obtained.

TABLE 17.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 2&2A–3&3A.

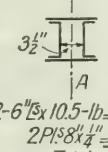
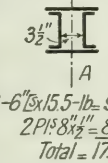
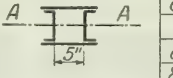
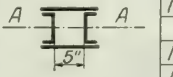
Type	Test no	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
2	11	Light Sections.	10.06	2.31	9′-7½″	50	33100		27400	
	41		10.15	″	″	″	34000	33200	28200	27500
	43		10.06	″	″	″	32400		26900	
	21		9.92	″	16′-4 3/8″	85	33700		29200	
	59		9.87	″	″	″	32300	32600	28400	28700
	65	2-6″ [s x 10.5-lb=6.18ª″	10.05	″	″	″	31700		28600	
		2 Pls 8″ x ¼″ = 4.00								
	32	Total = 10.18ª″	10.01	″	23′-1 3/16″	120	28300		25600	
	54		10.09	″	″	″	28700	29300	26300	27000
	70		10.07	″	″	″	30900		29000	
2A	101	Heavy Sections.	17.07	2.33	9′-8½″	50	32200		26000	
	124		17.07	″	″	″	32500	32300	27000	26800
	165		16.96	″	″	″	32300		27500	
	145		17.07	″	16′-6 1/16″	85	30700		27000	
	146		17.02	″	″	″	30000	30600	26000	26700
	151	2-6″ [s x 15.5-lb=9.12ª″	17.11	″	″	″	31000		27000	
		2 Pls 8″ x ½″ = 8.00								
	135	Total = 17.12ª″	17.04	″	23′-3 5/8″	120	28500		26500	
	136		17.02	″	″	″	28800	28100	26500	26300
	137		17.08	″	″	″	27000		26000	
3	1	Light Sections.	8.71	2.34	9′-9″	50	33800		28300	
	2		8.80	″	″	″	34300	34100	28700	28300
	42		8.71	″	″	″	34300		27900	
	14		8.71	″	16′-6 7/8″	85	32500		26900	
	87		8.69	″	″	″	31900	32400	29000	28000
	88	2-5″ [s x 6.5-lb=3.90ª″	8.60	″	″	″	32900		28000	
		2 Pls 9 1/2″ x 1/4″ = 4.75								
	24	Total = 8.65ª″	8.79	″	23′-4 13/16″	120	30600		27900	
	35		8.70	″	″	″	30200	30300	28600	28000
	47		8.74	″	″	″	30000		27500	
3A	105	Heavy Sections.	16.44	2.39	9′-11½″	50	29800		25500	
	168		16.45	″	″	″	29300	29500	25000	25300
	171		16.39	″	″	″	29500		25500	
	147		16.41	″	16′-11 1/8″	85	28000		26000	
	148		16.39	″	″	″	28000	28000	25500	25700
	150	2-5″ [s x 11.5-lb=6.76ª″	16.37	″	″	″	28000		25500	
		2 Pls 9 1/2″ x 1/2″ = 9.50								
	138	Total = 16.26ª″	16.50	″	23′-10 13/16″	120	28000		26500	
	139		16.45	″	″	″	26100	26900	26000	25700
	143		16.42	″	″	″	26600		24500	

TABLE 18.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 4&4A–8&8A.

Type	Test no.	Section.	Actual Area.	Radius of Gyration.	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
4	6	Light Sections.	11.69	2.38	9'–11"	50	35700		29900	
	36		11.69	"	"	"	36500	36900	33000	32000
	75	A [ ] A	11.69	"	"	"	38600		33000	
	13		11.66	"	16'–10⅝"	85	34000		30400	
	79		11.73	"	"	"	33900	34000	31500	31100
	80	2-8" ⊏ x 11.25-lb = 6.70ª"	11.64	"	"	"	34000		31500	
		1-8" I x 18.00-lb = 5.33								
	26	Total = 12.03ª"	11.69	"	23'–9⅝"	120	32000		30300	
	45		11.61	"	"	"	30700	31900	28200	30000
	49		11.68	"	"	"	33100		31600	
4A	102	Heavy Sections.	16.98	2.32	9'–8"	50	30300		23000	
	166		16.84	"	"	"	29000	29100	22500	22500
	169	A [ ] A	16.81	"	"	"	28000		22000	
	152		16.86	"	16'–5 3/16"	85	26000		22500	
	153		16.89	"	"	"	26400	26600	23500	23300
	154	2-8" ⊏ x 18.75-lb = 11.02ª"	17.00	"	"	"	27500		24000	
		1-8" I x 20.50-lb = 6.03								
	132	Total = 17.05ª"	16.98	"	23'–2 7/16"	120	23500		21500	
	133		17.07	"	"	"	23900	23900	22000	21700
	134		16.96	"	"	"	24200		21500	
8	9	Light Sections.	10.98	2.46	10'–3"	50	35900		30900	
	39		11.01	"	"	"	35100	35700	30500	31300
	78	A ⊏ ⊏ A	11.03	"	"	"	36200		32500	
	16		11.09	"	17'–5⅛"	85	31400		31400	
	81		11.05	"	"	"	32900	32800	30000	31000
	82	4-4" Z^S x 1 1/4" = 9.64ª"	11.11	"	"	"	34000		31500	
		1 Pl. 7 x 1/4" = 1.75								
	27	Total = 11.39ª"	11.06	"	24'–7 3/16"	120	29100		28200	
	46		11.11	"	"	"	29400	29700	25900	28200
	50		11.16	"	"	"	30400		30400	
8A	100	Heavy Sections.	25.78	2.51	10'–5½"	50	33000		29000	
	126		25.86	"	"	"	33500	32900	29500	29000
	127	A ⊏ ⊏ A	25.83	"	"	"	32200		28500	
	156		25.56	"	17'–9 3/8"	85	31300		31000	
	160		25.57	"	"	"	25300	31400*	20000	29700*
	161	4-4" Z^S x 5/8" = 22.20ª"	25.43	"	"	"	31500		28500	
		1 Pl. 7 x 5/8" = 4.38								
	162	Total = 26.58ª"	25.66	"	25'–1 3/16"	120	27500		24500	
	163		25.70	"	"	"	27000	27300	26000	25500
	164		25.68	"	"	"	27500		26000	

*Test no. 160 omitted in determining the averages.

TABLE 19.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 5, 5A & 5B.

Type	Test No.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
5	107	Light Sections.	9.62	1.98	8′–3″	50	38000		35000	
	122		9.65	"	"	"	38000	38000	34500	34700
	123	A — A	9.65	"	"	"	38000		34500	
	114		8.61	"	14′–0 5/16″	85	36000		34000	
	116		9.66	"	"	"	34000	34300	32500	32800
	119	8″ H Col. x 32-lb. = 9.17 □″	9.59	"	"	"	33000		32000	
	110		8.64	"	19′–9 5/8″	120	33900		33000	
	111		8.68	"	"	"	31000	32000	30200	31100
	108		8.61	"	"	"	31000		30000	
5A	106	Heavy Sections.	17.86	2.09	8′–8 1/2″	50	34000		30000	
	120		17.82	"	"	"	36100	35400	31000	31200
	121	A — A	17.86	"	"	"	36000		32500	
	115		18.05	"	14′–9 5/8″	85	33900		32000	
	117		17.73	"	"	"	32000	32300	30500	30600
	118	8″ H Col. x 62-lb. = 18.27 □″	17.77	"	"	"	31000		29300	
	109		17.76	"	20′–10 13/16″	120	30000		29000	
	112		17.97	"	"	"	30000	30000	30000	29300
	113		17.73	"	"	"	30000		29000	
5B	225	Extra Heavy Sections.	27.49	2.17	9′–0 1/2″	50	24000		19000	
	226		27.18	"	"	"	24600	25200	21100	19700
	182	A — A	27.50	"	"	"	27100		19000	
	227		27.28	"	15′–4 7/16″	85	22600		19600	
	228		27.27	"	"	"	23000	23500	20000	20200
	183	8″ H Col. x 91-lb. = 26.64 □″	27.45	"	"	"	24800		21000	
	223		27.50	"	21′–8 3/8″	120	21000		19000	
	224		27.54	"	"	"	21000	21300	18500	19200
	181		27.57	"	"	"	21800		20000	

TABLE 20.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 6&6A–10&10A.

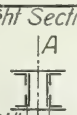
Type	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Unit Useful Limit Point.	Average Useful Limit Point.
6	10	Light Sections.	13.72	2.31	9'-7½"	50	31200		24200	
	40		13.78	"	"	"	31500	31600	23800	24200
	44		13.72	"	"	"	32200		24600	
	22		13.65	"	16'-4 3/8"	85	27800		23700	
	63		13.68	"	"	"	29500	29100	24500	24800
	64	1–I 10" x 25-lb. = 7.37a"	13.60	"	"	"	30100		26200	
		2 Pls. 11" x 5/16" = 6.88								
	31	Total = 14.25a"	13.73	"	23'-1 3/16"	120	26700		24100	
	52		13.75	"	"	"	27700	27200	24800	24500
	71		13.65	"	"	"	27300		24500	
6A.	103	Heavy Sections.	23.98	2.47	10'-3½"	50	31200		21000	
	125		23.75	"	"	"	32800	32100	22500	21800
	128		23.72	"	"	"	32400		22000	
	155		23.69	"	17'-5 15/16"	85	27200		23500	
	158		23.73	"	"	"	26800	26800	23000	23300
	159	1–I 10" x 35-lb. = 10.29a"	23.72	"	"	"	26400		23500	
		2 Pls. 11" x 5/8" = 13.75								
	129	Total = 24.04a"	23.64	"	24'-8 3/8"	120	24900		22000	
	130		23.83	"	"	"	24000	24800	23000	22500
	131		23.76	"	"	"	25500		22500	
10	3	Light Sections.	10.77	2.33	9'-8½"	50	36100		26000	
	4		10.80	"	"	"	35700	35800	27700	26800
	53		10.86	"	"	"	35600		26800	
	19		10.78	"	16'-6 1/16"	85	32500		26600	
	83		10.76	"	"	"	31900	32100	27000	26900
	86	2 Pls. 9" x 1/4" = 4.500a"	10.76	"	"	"	31900		27000	
		4 Ls. 2" x 2" x 1/4" = 3.760								
	23	2 Pls. 5 1/4" x 1/4" = 2.625	10.75	"	23'-3 5/8"	120	28400		27000	
	33	Total = 10.885a"	10.76	"	"	"	28300	28400	25800	26500
	34		10.75	"	"	"	28400		26700	
10A.	104	Heavy Sections.	18.60	2.34	9'-9"	50	31400		25500	
	167		18.52	"	"	"	32000	31800	26000	26000
	170		18.46	"	"	"	32100		26500	
	144		18.67	"	16'-6 7/8"	85	28000		24500	
	149		18.65	"	"	"	28000	28300	24500	24500
	157	2 Pls. 9" x 7/16" = 7.88a"	18.69	"	"	"	29000		24500	
		4 Ls. 2" x 2" x 7/16" = 6.24								
	140	2 Pls. 5 1/4" x 7/16" = 4.59	18.64	"	23'-4 13/16"	120	27000		24500	
	141	Total = 18.71a"	18.80	"	"	"	25900	26300	23500	24000
	142		18.65	"	"	"	26000		24000	

TABLE 21.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 1&1A–2&2A–4&4A.

Slenderness Ratio $\frac{l}{r}$ = 155.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
		Light Sections.								
	207		11.84	2.24	28′-11 3/16″	155	27000		26000	
1	218		11.69	"	"	"	25600	26200	25000	25200
	222	4 ls 5″x3″x 5/16″ = 9.60 a″ 1 Pl. 6″x 5/16 = 1.875 Total = 11.475 a″	11.68	"	"	"	26000		24500	
		Heavy Sections.								
	205		22.35	2.36	30′-5 13/16″	155	22200		21300	
1A	206		22.35	"	"	"	23200	22700	22400	21900
	208	4 ls 5″x3″x 5/8″ = 18.44 a″ 1 Pl. 6″x 5/8 = 3.75 Total = 22.19 a″	22.31	"	"	"	22700		21000	
		Light Sections.								
	215	3 1/2″	10.39	2.31	29′-10 1/16″	155	26100		25700	
2	217		10.42	"	"	"	27800	26500	26800	25700
	221	2-6″ ls x 10.5-lb = 6.18 a″ 2 Pls 8″x 1/4″ = 4.00 Total = 10.18 a″	10.46	"	"	"	25600		24700	
		Heavy Sections.								
	214	3 1/2″	16.75	2.33	30′-1 1/8″	155	24700		24000	
2A	216		16.87	"	"	"	25000	24900	23000	24000
	219	2-6″ ls x 15.5-lb = 9.12 a″ 2 Pls 8″x 1/2″ = 8.00 Total = 17.12 a″	16.81	"	"	"	25000		25000	
		Light Sections								
	211		11.81	2.38	30′-8 7/8″	155	24000		21000	
4	212		11.93	"	"	"	23200	23600	23000	22300
	220	2-8″ ls x 11.25-lb = 6.70 a″ 1-8″ I x 18.00-lb = 5.33 Total = 12.03 a″	11.84	"	"	"	23500		23000	
		Heavy Sections.								
	209		16.74	2.32	29′-11 5/8″	155	24000		24000	
4A	210		16.67	"	"	"	23000	23300	23000	23300
	213	2-8″ ls x 18.75-lb = 11.02 a″ 1-8″ I x 20.50-lb = 6.03 Total = 17.05 a″	16.33	"	"	"	23000		23000	

The Committee has carefully considered the application of this method to the stress-strain diagrams of full-size columns. Tables 16 to 21 give the U. L. P. (Useful Limit Points) for the column tests, together with the average for the three of each series, also the ultimate strength, so that comparison may be made between the values of the U. L. P. and the ultimate strengths, and Plates XXXIII, XXXIV, and XXXV are given as samples to illustrate the method used by the Committee in determining the U. L. P. values. To publish all the diagrams used in these investigations would cost the Society a large sum of money, and the Committee is of the opinion that such publication in this report is not necessary. At the completion of the work of the Committee, the tracings of the diagrams will be filed in the Engineering Societies Library, and they can be reproduced at a nominal expense for those engineers who desire to make a special study of this phase of the work.

It will be seen from a study of the tables of U. L. P. values that this method offers an extremely valuable indication of the strength of full-size columns. It will be seen, further, that the shape of the cross-section of the columns, as covered by the programme of the Committee, is of minor importance when considered in connection with the strength of the material, as shown by the U. L. P. values.

As mentioned above, and as will be seen from Tables 11 to 15, the specimen coupon tests are not complete. The Bureau of Standards planned to carry on a very elaborate supplementary investigation during 1917. For some time, however, the testing capacity of the Bureau has been given over to testing materials for war purposes. The tests here published show all that Dr. Stratton can promise until the close of the War. The Committee feels that the results shown by these tests would not be changed materially by additional tests in the series, and, realizing the demand for information on the subject, desires to make a final report based on the available data.

The data mentioned above, namely Tables 1 to 21, inclusive, showing the values of the U. L. P. (Useful Limit Points), furnish all the information which the Committee has obtained in accordance with the first item of its commission—to determine the ultimate strength of steel columns and struts. In making this as a report, the Committee has not attempted to correlate these tests with the older ones shown in its report of 1910. Some of the columns given in the 1910 report were tested many years ago, when it was not thought necessary to have such accurate measurements as have been made in the present series. To attempt to correlate the older column tests with the present ones would, in the opinion of the Committee, be of no value except as a general speculation.

It will be seen at once from the study of the foregoing tables that the record is unsatisfactory, on account of the large variation in ultimate strengths and U. L. P. (Useful Limit Points) of the columns. Some

of this variation may be brought about by the shape of the column, but most of it can be explained by the condition of the material. Plate XXXVI is a summary of the curves of all the U. L. P. values. A surprising feature of these curves is the variation in U. L. P. of 15 000 lb. per sq. in. between columns of Types 5 and 5*B*. The average U. L. P. for $\frac{l}{r} = 50$, for columns of Type 5, light section, is 34 700 lb. per sq. in., and for Type 5*B*, extra heavy section, it is 19 700 lb. per sq. in. The lower value is but 57% of the higher, and there is a variation of 28% from the mean. These extreme variations may be abnormal, but it must be remembered that the tests were made on material specially chosen, and supposedly the best that the mills could produce.

On account of this large range in the results, it is not possible for the Committee to execute satisfactorily the first portion of its commission, which was to determine the ultimate strength of steel columns. The Committee does not believe it is safe to take the average strength of all the columns which the Bureau of Standards has tested as a basis for the design of columns where there is uncertainty as to the quality of the material. In order to make a constructive report, therefore, it is necessary to consider the second part of the Committee's commission, namely, to determine the safe working value for columns, and to make this a safe working value conservative enough to cover the wide range in the results shown by the Committee's programme.

In considering the safe working value for a column, it is necessary to take account of a number of uncertain and sometimes unknown factors. Some of these might be enumerated as follows:

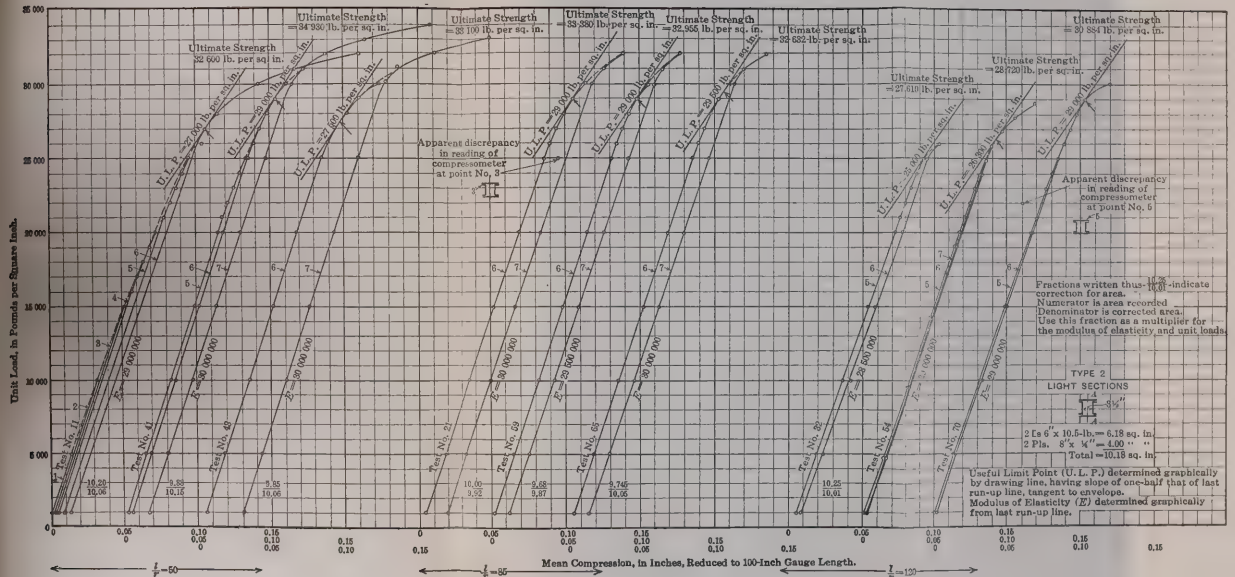
(1) Material:
  (*a*) Quality of steel.—High and low carbon.—Amount of work received.
  (*b*) Uniformity.—Segregations.—Hard and soft spots.
  (*c*) Workmanship.—Initial bends.—Inadequate or poor riveting.

(2) Condition of loading:
  (*a*) End bearing.—Pin or square ends.—Ends not milled perpendicular to axis.
  (*b*) Eccentricity.—Load not applied at axis of column.
  (*c*) Transverse loading.—Combined beam and column action.
  (*d*) Repeated stresses.—Loads repeated and perhaps alternated many times.

(3) Use:
  (*a*) Overloading.—Impossibility of always determining exact loading.
  (*b*) Secondary stresses.— Sometimes a considerable proportion of the primary stress.

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1456.
FINAL REPORT OF
SPECIAL COMMITTEE ON
STEEL COLUMNS AND STRUTS.

1676


Unit Load, in Pounds per Square Inch.
Ultimate Strength = 34 930 lb. per sq. in.
Ultimate Strength = 32 600 lb. per sq. in.
Ultimate Strength = 33 100 lb. per sq. in.
Ultimate Strength = 33 380 lb. per sq. in.
Ultimate Strength = 32 955 lb. per sq. in.
Ultimate Strength = 32 682 lb. per sq. in.
Ultimate Strength = 30 884 lb. per sq. in.
Ultimate Strength = 28 720 lb. per sq. in.
Ultimate Strength = 27 410 lb. per sq. in.
U. L. P. = 27 600 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 27 400 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 27 600 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
Apparent discrepancy in reading of compressometer at point No. 3
Apparent discrepancy in reading of compressometer at point No. 5
E = 30 000 000
E = 29 000 000
Test No. 44
Test No. 49
Test No. 21
Test No. 69
Test No. 66
Test No. 22
Test No. 54
Test No. 70
Mean Compression, in Inches, Reduced to 100-Inch Gauge Length.
l/r = 50
l/r = 85
l/r = 120

Fractions written thus $\frac{10.28}{10.01}$ indicate correction for area.
Numerator is area recorded
Denominator is corrected area.
Use this fraction as a multiplier for the modulus of elasticity and unit loads.

TYPE 2
LIGHT SECTIONS

2 ls 6" x 10.5-lb. = 6.18 sq. in.
2 Pls. 8" x ¼" = 4.00 " "
Total = 10.18 sq. in.

Useful Limit Point (U. L. P.) determined graphically by drawing line, having slope of one-half that of last run-up line, tangent to envelope.
Modulus of Elasticity (E) determined graphically from last run-up line.

(*c*) Deterioration or corrosion.—Exposure to corrosive elements.

(*d*) Facilities for inspection.—Forms of columns which may be frequently and easily examined.

(*e*) Warning of failure.—Practically no warning, as compared with tension members.

(*f*) Gravity of injury caused by failure.—Damage to life and property in some cases, or only temporary inconvenience in other cases.

(*g*) Cost of replacement.

In the early days of the iron and steel industry, it was the custom of engineers to adopt a working unit stress for tension members of one-quarter of the unit ultimate specimen tensile strength, and they spoke of a factor of safety of four. When we consider the distortions produced by the stretching of tension members after being strained above the yield point, and that manifest yielding and failure occur in columns when the stress reaches about one-half of the specimen ultimate strength in tension, it is evident that the factor of safety obtained by this older method was nearer two than four.

In a structure having both tension and compression members, the desideratum in determining a factor of safety is to obtain a working stress so that all parts of the structure have an equal capacity to resist the applied loadings.

Your Committee has made no original investigations of the strength of full-size riveted tension members, and, therefore, cannot make as definite a comparison with full-size riveted columns as would be desirable. It may be stated, however, that the usual working stress in tension is approximately one-half the elastic value of the metal, and the Committee assumes that, in view of all the factors mentioned above, columns should have a safety factor of at least two, based on the U. L. P., in order to be on a parity with tension members.

The average U. L. P. of all the column tests in the Committee's programme for slenderness ratios, $\frac{l}{r} = 50$ and $\frac{l}{r} = 85$, is 27 200 lb. per sq. in. The U. L. P. for the extra heavy section, Type 5*B*, slenderness ratio $\frac{l}{r} = 50$, which is the lowest value observed, is 19 700 lb. per sq. in., which is 28% below the average, and this appears to be too wide a margin of under-run for safety. It would seem to be necessary, therefore, in recommending a working stress, to assume a U. L. P. lower than the average of all the tests. If we take as a safe assumption the mean between the lowest value and the average value, this mean will be 23 500 lb., or approximately 24 000 lb. per sq. in. The factor of two applied to 24 000 lb. will give a safe working value

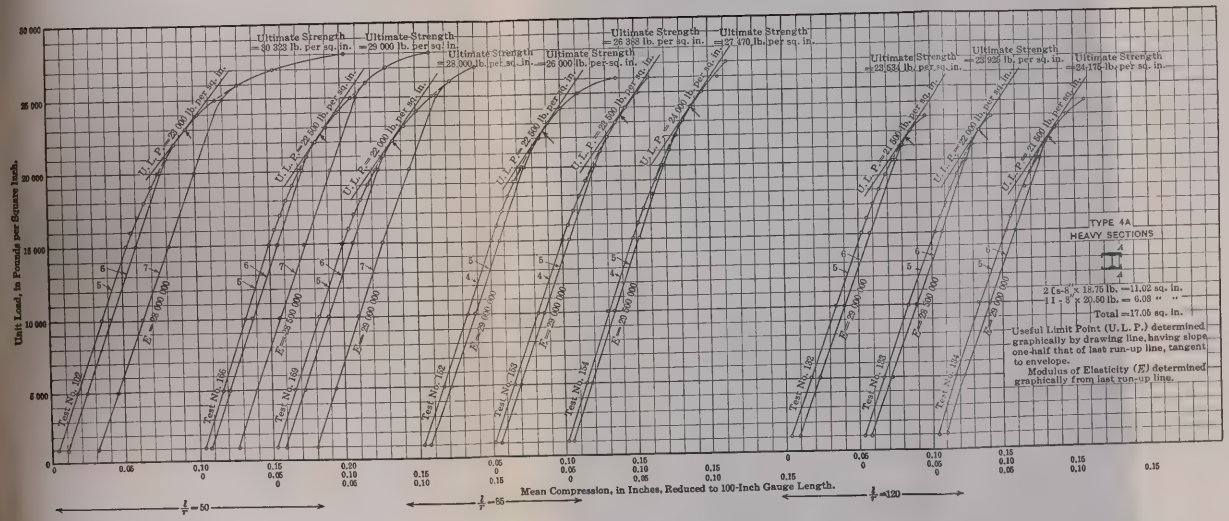
of 12 000 lb. per sq. in. for columns, which stress the Committee would recommend. In this recommendation it is assumed that we are dealing only with static loads, and that a percentage of the static stress will be added to cover the impact due to moving loads.

Lacking further experimental data, the Committee regards it as unwise to assume a higher working stress than 12 000 lb. per sq. in. for columns in which the ordinary grade of structural steel (60 000 lb. ultimate tensile strength desired), is specified. It is, of course, impracticable to know in advance the precise U. L. P., or any other factor of strength, which the metal used may develop. It would be possible to specify the desired value, and, in important structures, to inspect the material with sufficient care to insure the rejection of all which failed to come up to the specification. Later in the report, suggestion is made of a method whereby those engineers who have ample facilities for testing materials may adopt such working stresses as are warranted by the special material tests. It must be remembered, however, that a large portion of the material which is classed as structural steel is furnished on the manufacturer's specifications, and, in view of the Committee's findings, probably most structural steel is incompletely tested.

The Committee would recommend that this working stress of 12 000 lb. be used for columns up to a slenderness ratio of $\frac{l}{r} = 80$, and, above this slenderness ratio, the Committee would reduce the working stress to allow for uncertainties. The Committee realizes that the results as given in its programme show that the slenderness ratio has a comparatively small effect, up to values of $\frac{l}{r} = 120$. It must be remembered, however, that the tests were made by the Bureau of Standards under extremely favorable conditions, the ends of the column being scraped so as to give a bearing precisely perpendicular to the axis of the column. The Committee would recommend a working stress of 8 000 lb. per sq. in. for a slenderness ratio of $\frac{l}{r} = 120$, and that the working stresses for slenderness ratios between 80 and 120 be determined by interpolation.

The Committee realizes that the working value recommended for short columns is lower than that given by the American Railway Engineering Association formula, which has been in use for a number of years. Originally this formula was $\frac{P}{A} = 16\,000 - 70\frac{l}{r}$ and, later, the upper section was truncated to a maximum working stress of 14 000 lb. The Committee feels, in view of its studies in regard to the U. L. P., that there is no warrant for high working stresses in short columns.

PLATE XXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1456.
FINAL REPORT OF
SPECIAL COMMITTEE ON
STEEL COLUMNS AND STRUTS.


Unit Load, in Pounds per Square Inch.
Ultimate Strength = 30 323 lb. per sq. in.
Ultimate Strength = 29 000 lb. per sq. in.
Ultimate Strength = 28 000 lb. per sq. in.
Ultimate Strength = 26 000 lb. per sq. in.
Ultimate Strength = 26 368 lb. per sq. in.
Ultimate Strength = 27 470 lb. per sq. in.
Ultimate Strength = 23 534 lb. per sq. in.
Ultimate Strength = 23 925 lb. per sq. in.
Ultimate Strength = 24 175 lb. per sq. in.
U. L. P. = 29 000 lb. per sq. in.
U. L. P. = 27 500 lb. per sq. in.
U. L. P. = 25 000 lb. per sq. in.
U. L. P. = 22 000 lb. per sq. in.
U. L. P. = 24 000 lb. per sq. in.
E = 30 000 000
E = 29 000 000
E = 28 000 000
E = 25 000 000
E = 24 000 000
E = 22 000 000
Test No. 192
Test No. 189
Test No. 154
Test No. 134
TYPE 4A
HEAVY SECTIONS
2 [s-8" × 18.75 lb. = 11.02 sq. in.
1 I - 8" × 20.50 lb. = 6.03 " "
Total = 17.05 sq. in.
Useful Limit Point (U. L. P.) determined graphically by drawing line, having slope one-half that of last run-up line, tangent to envelope.
Modulus of Elasticity (E) determined graphically from last run-up line.
Mean Compression, in Inches, Reduced to 100-Inch Gauge Length.
l/r = 50
l/r = 85
l/r = 120

Argument might be made that higher working values than those recommended by the Committee have been in use for long periods of years, without failure, but the Engineering Profession does not need to be reminded of accidents which have been caused by the failure of columns. Furthermore, it is evident that the present processes used in the manufacture of steel do not produce a uniform material, and that the present methods of testing do not give a correct indication of the compressive strength of the material.

TABLE 22.—Summary of Results of Mill Tests and Bureau of Standards Tests.

Type.	Mill Specimen Tests made at Pittsburgh.			Bureau of Standards Specimen Tests.			Average U.L.P. of full-size columns $\frac{l}{r} = 50$ and 85.	Average ultimate strength of full-size columns $\frac{l}{r} = 50$ and 85.
	Average drop of beam.	Average ultimate strength	Number of tests.	Average U. L. P.	Average ultimate strength	Number of tests.		
1	34 000	58 900	7	33 100	59 000	8	28 300	32 000
1A	38 300	59 500	7	29 800	56 200	2	25 000	28 600
1B	36 400	59 600	4	31 100	57 000	1	26 200	27 700
2	36 500	57 700	5	32 500	58 800	8	28 100	32 900
2A	36 900	59 700	4	30 200	58 400	6	26 800	31 400
3	39 800	55 600	5	34 100	55 400	4	28 200	33 300
3A	37 900	59 200	2	.....	.....	.....	25 500	28 800
4	36 500	58 100	5	33 800	58 000	13	31 600	35 400
4A	37 700	59 400	4	27 100	59 700	13	22 900	27 900
5	.....	.....	.....	37 500	61 500	2	33 800	36 200
5A	.....	.....	.....	33 300	57 900	3	30 900	33 800
5B	.....	.....	.....	22 600	56 400	12	20 000	24 400
6	34 600	57 600	9	.....	.....	.....	24 500	30 400
6A	38 300	60 400	3	.....	.....	.....	22 600	29 500
7	34 600	59 400	5	33 500	61 600	3	30 000	32 500
8	39 200	56 300	6	.....	.....	.....	31 200	34 200
8A	37 600	58 700	3	.....	.....	.....	29 400	32 300
10	37 600	58 100	16	.....	.....	.....	26 800	34 000
10A	37 600	61 700	3	.....	.....	.....	25 200	30 100

The difference between the strength of the material as indicated by the mill tests made in the ordinary commercial manner and the same material when fabricated into columns is strikingly shown by Table 22. It will be noted from the portion of the table giving the values for the specimens tested at the mill that neither the "yield points", as indicated by the drop of the beam, nor the ultimate strengths, bear any consistent relation to the U. L. P.'s, or ultimate strengths,

of the full-size columns. In most cases, material which gave the lower strength in the full-size columns gave the higher "yield point", as indicated by the mill tests, and, on the other hand, the U. L. P.'s of the specimens, as determined by the Bureau of Standards, bear a very definite and constant relation to the U. L. P.'s of the full-size columns. While the studies of the Committee point to a method whereby the strength of material for columns may be correctly determined, until some such method has been generally adopted, the Committee must recommend a conservative safe working value.

The Committee has not attempted to determine safe working values for columns with a slenderness ratio above $\frac{l}{r} = 120$. Some tests have been made and recorded for a slenderness ratio of $\frac{l}{r} = 155$ which may serve as a guide to engineers, but the Committee does not feel that these data are sufficient to carry its recommendation higher than a slenderness ratio of $\frac{l}{r} = 120$.

The Committee's report covers only sections designed to avoid the necessity of latticing or battens, and tested with square ends. It has not been possible to consider the effect of different arrangements of the ends. Details, such as lattice bars, tie-plates, batten-plates, and pin-plates, have not been considered, because the Committee felt that the more important factor was the strength of the material in the columns, and that the question of details could be left until later. The Committee would refer engineers to the reports of the Committee on Iron and Steel Structures of the American Railway Engineering Association, which has worked in co-operation with your Committee, and has considered some of the details mentioned above.

Your Committee's report covers only one grade of steel, which is the ordinary structural grade, with a desired ultimate tensile strength of 60 000 lb. per sq. in. Your Committee has not considered, except in a very general way, the question of transverse loadings, repeated loadings, and long-time tests, and these features will have to await the accumulation of additional data.

The Committee realizes the demand of many engineers for a scientific, rational column formula. Numerous theoretical formulas have been promulgated from time to time, all of them based on entirely elastic material of uniform grade. At the present time it seems useless to the Committee to attempt to write a rational column formula, from a theoretical standpoint, with the possibility of variations of 28% in the strength of the material.

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXIII, No. 1456.
FINAL REPORT OF
SPECIAL COMMITTEE ON
STEEL COLUMNS AND STRUTS.

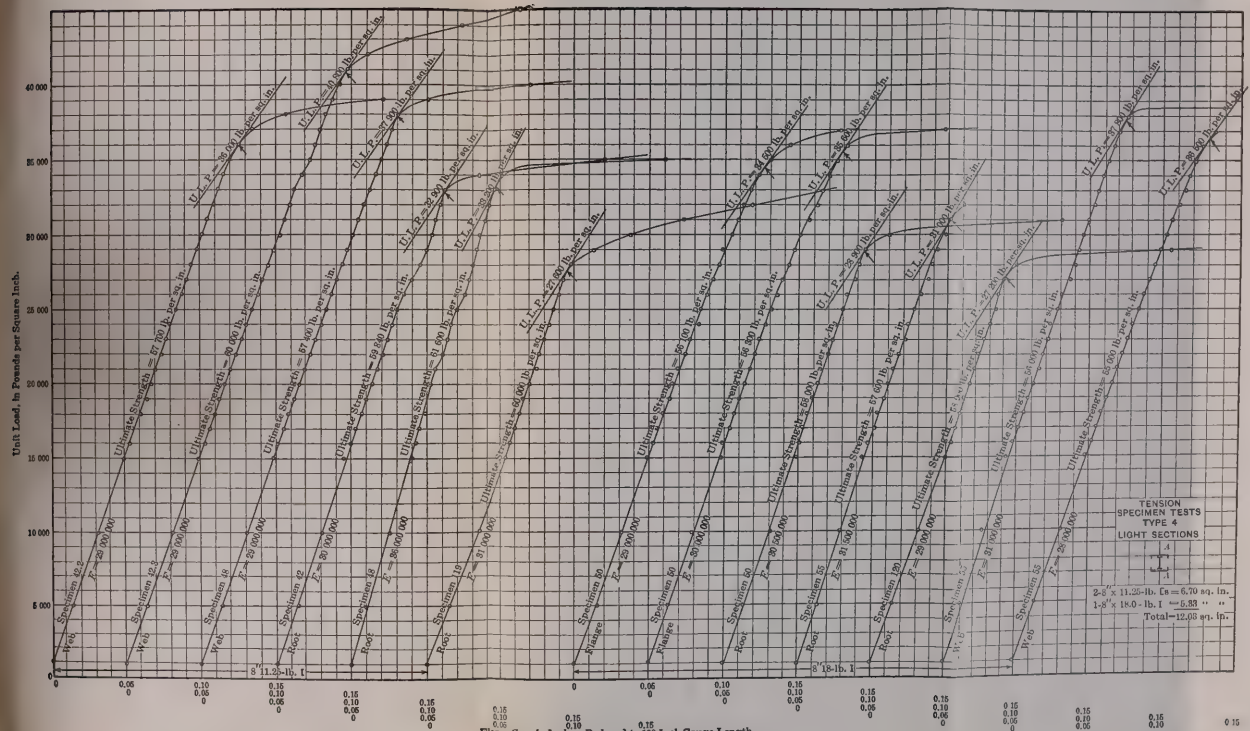

Unit Load, in Pounds per Square Inch.
Elongation, in Inches, Reduced to 100-Inch Gauge Length.
TENSION SPECIMEN TESTS TYPE 4 LIGHT SECTIONS
2-8" x 11.25-lb. I a = 6.70 sq. in.
1-8" x 18.0-lb. I = 5.33 " "
Total=12.03 sq. in.
8" 11.25-lb. I
8" 18-lb. I

TABLE 23.—Summary of Results, Showing Relation Between Full-Size Column Tests and Specimen Tests.

Type.	U. L. P's of Full-Size Columns. Each value is the average of three test results.			Weighted average U. L. P's of all tension and Comp. Spec. tests.	Percentage which U. L. P's of full-size column tests are below U. L. P's of Spec. tests.	Total number of specimen tests.	Average U. L. P's of tension tests on web specimens.	Percentage which U. L. P's of full-size column tests are below U. L. P's of web specimen tests.	Number of web tests on web specimens.
	$\frac{l}{r} = 50$.	$\frac{l}{r} = 85$.	Average of $\frac{l}{r} = 50$ and 85.						
1	28 800	27 800	28 300	33 400	15.3	13	33 600	15.8	5
1A	24 700	25 300	25 000	28 400	12.0	5	29 800	16.1	2
2	27 500	28 700	28 100	32 800	14.3	14	33 800	16.9	6
2A	26 800	26 700	26 800	32 100	16.5	6	32 500	17.5	3*
3	28 300	28 000	28 200	35 300	20.1	5	38 100	26.0	2
4	32 000	31 100	31 600	36 000	12.2	18	37 800	15.4	5
4A	22 500	23 300	22 900	27 200	15.8	21	25 600	10.5	6
5	34 700	32 800	33 800	37 200	9.1	3	37 000	8.7	1
5A	31 200	30 600	30 900	33 400	7.5	3	32 500	4.9	1
5B	19 700	20 200	20 000	23 200	13.8	12	24 900	19.7	3
7	30 600	29 400	30 000	33 900	11.5	6	33 200	9.6	2
				Averages	13.5			14.7	

* Test No. 66 omitted in taking averages.

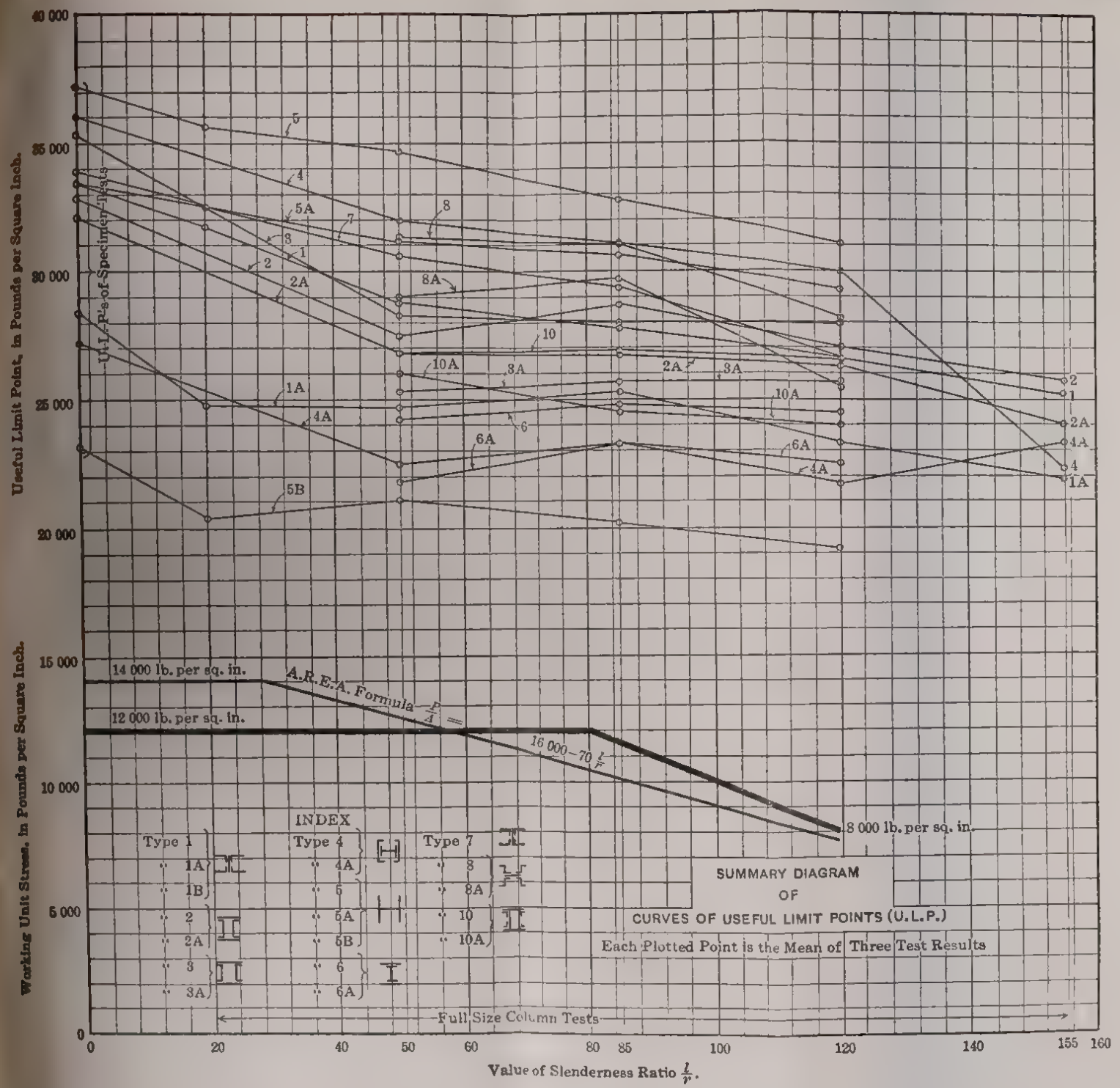
The careful selection of specimens located by the inspector, and the testing of these with extensometers, means an increased cost of the material, both to the rolling mill and to the purchaser. The Committee believes, however, that the resultant saving and the assurances of safety for large tonnages and important structures would warrant such increased expense. Possibly such inspection, if extensively carried out, would improve the methods of manufacture of steel, thereby producing a more uniform material.

To those engineers who desire to undertake such inspection, the Committee would recommend the study of Table 23, which shows the comparison between the U. L. P.'s of the specimen tests, and the U. L. P.'s of the full-size tests. As before stated, the Bureau of Standards has not been able to supply the Committee with the full quota of specimen tests. It will be seen that, where only a few specimen tests were furnished, the complete record of the material, so far as relates to the condition in the flange and the root, cannot be satisfactorily averaged, but the Committee is giving all the information that it has at hand. Some of the specimen tests were made in tension and some in compression, and these tests from different parts of the cross-sections of the columns were weighted in proportion to arbitrarily assumed areas. The data are too meager to warrant a recommendation as to the use of the average of these percentages in the design of future columns, but they may serve as a guide to those engineers who desire to begin investigations. As most specimen tests are made from the long legs of angles and the webs of I-beams and channels, on account of the convenience in securing the specimens, the table giving the percentage of the U. L. P. of full-size columns below web specimens should afford the best guide as to what might be the probable U. L. P. strength of full-size columns after the specimen U. L. P. was known.

As stated in the beginning of this report, your Committee has been in service for nine years, and it is the feeling of its members that collateral lines of investigation can be carried out by the appointment of a new Committee, rather than by the continuance of the old one.

Complete records of the work of the Committee will be filed in the Engineering Societies Library. At the close of the War, the Bureau of Standards may be in a position to complete the programme of testing the coupon material set aside for specimen tests, and the Committee has the assurance of Dr. Stratton that the record of this result will also be available for the files of the Library.

Your Committee desires to express thanks to the Bureau of Standards, which provided the tests on which this report is based, and to Dr. S. W. Stratton, Director, and his assistants, who made many investigations and suggestions, for their loyal co-operation and support.


Useful Limit Point, in Pounds per Square Inch.
Working Unit Stress, in Pounds per Square Inch.
U.L.P.'s of Specimen Tests
40 000
35 000
30 000
25 000
20 000
15 000
10 000
5 000
0
14 000 lb. per sq. in.
A.R.E.A. Formula $\frac{P}{A} = 16\,000 - 70\,\frac{l}{r}$
12 000 lb. per sq. in.
8 000 lb. per sq. in.
INDEX
Type 1
1A
1B
2
2A
3
3A
Type 4
4A
6
5A
5B
6
6A
Type 7
8
8A
10
10A
SUMMARY DIAGRAM
OF
CURVES OF USEFUL LIMIT POINTS (U.L.P.)
Each Plotted Point is the Mean of Three Test Results
Full Size Column Tests
0
20
40
50
60
80
85
100
120
140
155
160
Value of Slenderness Ratio $\frac{l}{r}$.

Believing that the foregoing report completes the work which the Committee can at present perform to advantage, it now asks to be discharged.

For the Committee,

LEWIS D. RIGHTS,
Chairman.
C. W. HUDSON,
Secretary.

COMMITTEE:

JAMES H. EDWARDS,
C. W. HUDSON,
CHARLES F. LOWETH,
RALPH MODJESKI,
LEWIS D. RIGHTS,
GEORGE F. SWAIN,
EMIL SWENSSON,
J. R. WORCESTER.

NOVEMBER 22D, 1917.

DISCUSSION

Mr. Prichard.

HENRY S. PRICHARD,* M. AM. SOC. C. E. (by letter).—The Committee is to be congratulated on its fine presentation of the results of the tests conducted by the Bureau of Standards. As a progress report, it is admirable, but the writer thoroughly endorses the sentiments which Clement E. Chase, Jun. Am. Soc. C. E.,† has expressed, in a letter just received from him, as follows:

"In reading what the Column Committee of the American Society of Civil Engineers are to present next week as their Final Report, I fail to see why they should consider their task at an end. They seem to have answered so few of the questions that have been raised in recent years as to column action, and to have passed so lightly over what has been done in the past in the way of experiment, analysis and discussion. I think their concept of a Useful Limit Point a good one, getting away from the controversial elastic limit, but I do not see that they give any evidence that what they call the U. L. P. is the useful limit.

"I expect to get down to the meeting next week, and have been wondering what would develop there. I wish that there were some way of continuing the Committee in existence, changing its personnel if the old members are tired of their work, for it seems hard to get Special Committees appointed in the Society. It would not be possible to do much in the way of testing during the war, but some planning of the next series of tests could be accomplished, and there would not be the effect of calling this report the Final and best that can be done after nine years of deliberation. If you agree and think that anything could be accomplished by discussion of the Report, or letter, in time for action at the Annual Meeting, I wish you could see your way to start something."

Why should the Committee consider its work at an end? There are many open questions that need further investigation before they can be answered; and there is much that has been done in the past in the way of experiments, analyses, and discussion which the Committee has passed over lightly but of which it is desirable that a Final Report should include an epitome and an appreciation.

The point in testing where the rate of deformation becomes double the elastic rate has been adopted by the Committee as useful for practicable purposes and termed the "Useful limit point". "Double rate of deformation point" would be a better name, as it is accurately descriptive and does not commit the Committee to a characterization of the significance of the point as an indication of strength.

It has not been shown that the metal in a structural member ceases to have structural value when the rate of deformation of the member becomes double the elastic rate. In fact, Test No. 102, used by the

* Pittsburgh, Pa.
† Now Assoc. M. Am. Soc. C. E.

Committee to illustrate the determination of the point where such a rate of deformation occurs, shows that, at the cost of a slight permanent set, the column became seemingly elastic to a higher point. Mr. Prichard

As regards specimen tests, the results tabulated by the Committee do show that the averages or means of the "Useful limit points" or "Double rate of deformation points", as the writer prefers to call them, of the coupons of the material of which the columns are composed, indicate, approximately, the ultimate strength of those columns which have a slenderness ratio of $\frac{l}{r} = 50$. The average ultimate strength of the thirty-three columns compared was 32 670 lb. per sq. in., though the average of the means of the "Double rate of deformation points" of the specimens was 32 080 lb. per sq. in.; and in no case was the ultimate strength of the columns less than the mean of the "Double rate of deformation points" by more than 3½ per cent. This close relationship, which the report of the Committee shows, between the mean of the "Double rate of deformation points" of the specimens representative of a column, and the ultimate strength of the column, is a valuable contribution to engineering knowledge; but there remain many important questions, some of which can probably be answered by a study of the results of past experiments. The work of the Committee could be continued with advantage to the Profession.

GUSTAV LINDENTHAL,* M. AM. SOC. C. E. (by letter).—This final report, giving the results of investigations and tests continued through seven years, will no doubt be regarded by structural designers as an authoritative guide in the proportioning of steel columns of such forms and in such cases as those to which the conclusions of the Committee are applicable. Mr. Lindenthal.

The Committee was entirely justified in giving warning (page 1626) that in considering the safe working value for a column it is necessary to take account of a number of uncertain and sometimes unknown factors. The Committee then enumerates fourteen such factors, divided into three groups (Material Condition of Loading, and Use). One factor, however, is not mentioned, and, like any of the others, it may have a deteriorating effect on the strength of the column under certain conditions; that is, the factor of the rivet holes when the column section is made up of different steel shapes riveted together into one column.

The rivet holes in columns quite frequently do not match accurately, notwithstanding the usual reaming to bring the holes to a fit. When the section is made up of three or more thicknesses, the holes in the inside plates, after reaming, may be oblong rather than round. The rivet, after driving and upsetting, may fit the hole tightly near each rivet head, but not always in the middle of the rivet. Moreover, the

* New York City.

Mr. Lindenthal.

diameter of the rivet after cooling may be slightly smaller than the rivet hole. Most so-called tight rivets can be pushed out of the holes easily after one or the other rivet head is cut off. It cannot be claimed, therefore, that the rivets fill the holes so completely as to equal the solid metal before the holes were punched or drilled out. The rivet in the hole, also, frequently has scale on it, forming a hard film without strength. If that film is only $\frac{1}{2000}$ in. thick, then the rivet would be $\frac{1}{1000}$ in. smaller than the hole. Even with good workmanship, the fit for part of a tight rivet is more frequently less than $\frac{1}{1000}$ in. It is evident that rivet holes weaken a compression section as well as a tension section.

A plate, for instance, 12 in. wide and 1 in. thick, with four rivet holes $\frac{3}{4}$ in. in diameter, on a horizontal section has a net section of 9 sq. in., which is its value for tension; but, for compression, the universal practice is to assume full 12 sq. in. as resisting the compressive stress. Have we any good reasons or tests anywhere to show that this assumption is justified?

It is obvious that the rivet hole must flatten before the metal in the rivet can participate in the compressive strength of the cross-section. It means (assuming a modulus of elasticity of 30 000 000) that the unit stress on the steel of the net cross-section must exceed 30 000 lb. before the rivet hole will flatten $\frac{1}{1000}$ of its diameter and before the rivet can take up a share of the compressive strain in the column, which, by that time, would be 22 500 lb. per sq. in. on the unperforated cross-section. The elastic limit of the steel will thus be passed on the net section long before it is reached in the unperforated cross-section. Failure of the column by crushing or by buckling may thus be initiated in the over-stressed metal of the net section, and thus the low compression value based on the cross-section of riveted columns may be explained.

If this reasoning is correct, then the cross-section of a column is weakened in proportion to the rivet holes. That view has induced the writer, in his practice, to formulate the rule that the compressive stress shall be computed, not on the cross-section of the column, but on its net section, similar to the practice for riveted tension members. It is certainly on the side of greater safety, more particularly for the reason that failure of a steel frame structure (designed by the usual specifications), if loaded to destruction, would surely come first through the buckling of the compression members before any tension member would give way. The general belief that our framed steel bridges have a factor of safety of from 3 to 4 needs to be corrected. Frequently, it is not more than 2, but that includes the dead load. It is an ample margin for heavy structures, but hardly enough for lighter ones.

Mr. Lindenthal.

The writer has attempted a series of tests, with the object of clearing up this point of the weakening of columns by rivet holes, and for these has used rolled steel pipe sections, some unperforated and others perforated and the holes filled with rivets. The tests were made at the laboratory of the United States Bureau of Standards in Pittsburgh, but were not concluded. Such tests, however, should be continued until this question is definitely cleared up.

One of the most remarkable points brought out in the tests by the Committee is the great difference in the compressive strength of the Bethlehem **H** sections, referred to in the report as Sections 5, 5*A*, and 5*B* (Tables 15 and 19). That the U. L. P. should be 34 700 for the light section and only 19 700 for the extra heavy section of the same type (page 1626) and for the same slenderness ratio (50), may be astonishing until we consider that the lower temperature at which the lighter section was very probably finished in rolling, as compared with the heavier section of the same type, must have a great influence on its value as a column. The lighter section is thus provided with a hard skin, possibly not more than $\frac{1}{20}$ in. thick, which is stronger than the metal underneath. That harder metal has a higher elastic limit, and thus can better resist the fiber strains from bending and buckling before final collapse. This view derives confirmation from the fact mentioned by the Committee (page 1612) that in short lengths and for the same ratio of l to r (when the metal is exposed almost solely to block action) the differences in compressive strength disappeared.

The recommendation of the Committee that the testing of steel columns should be continued is well founded. We need to know more about the behavior of riveted column sections as compared with solid sections of the same size. Measurement with the extensometer would show whether the compressive strain in columns is uniform for the entire length or whether the steel is stressed higher in the plane of the rivet holes. To establish this fact satisfactorily, the rivet spacing should not zig zag, but, experimentally, be on straight lines at right angles to the axis of the column. Such measurements should be made and compared on solid, rolled, unperforated, tube sections, and on others which have been perforated.

The definition and establishment of the U. L. P. (Useful Limit Point) by the Committee is commendable and worthy of general adoption. Also, the use of round figures for compressive unit stresses (similar to the practice for tension unit stresses) in place of the senseless refinement (by 10 and 20 lb. per sq. in., as given in some tables) derived from the too pedantic use of compression formulas.

Mr. Fowler.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E. (by letter).—The Committee deserves more than a perfunctory vote of thanks for the

* New York City.

Mr. Fowler.

wonderful devotion its members have shown in this most essential experimental work. Yet it is apparent that the scope of the inquiry must be broadened if future investigations are to reach the desired result.

There must be a determination of the essential details of built-up columns necessary to develop the greatest possible percentage of main sections; a determination of the principal riveting, together with the effect of riveting as respects the deduction for rivet holes; a means of procuring high steel of a uniform quality; and, lastly, an accurate means of determining the stresses due not only to live and dead load, but including impact, secondary stresses, temperature stresses, and initial stresses in the steel, to the end that there may be used a very much higher unit stress than the one proposed, thus reaching an economy and efficiency in bridge design from which engineers have departed so widely in the past twenty years.

When the writer was Bridge Engineer of the Hocking Valley Railway, in 1887, the pendulum was just beginning to swing back from the era of light loads and high unit stresses, which had resulted in bridges of the lightest type, where "the pins only held by the grace of God, and where the wind stresses had to slide down the end posts." The loading then beginning to be used was thought to be phenomenal—two 92.3-ton consolidation engines and unit stresses below 10 000 lb. per sq. in. were discussed. During the period from 1890 to 1898, in which the writer was Chief Engineer of the Youngstown Bridge Company, the pendulum swung far, and Cooper's E-40 and other heavy loadings came into vogue; and though engineers still clung to their factors of safety, unit stresses as low as 6 000 or 8 000 lb. per sq. in., based on factors of ignorance, were used on the primary members of bridges.

Thus, up to the present, has been kept up the increase of loadings and the reduction of unit stresses, until engineers have wandered far from that efficiency which they must have, under the new economic and trade conditions that are dawning. Engineers must know the stresses with certainty, they must have the best and most uniform high steels, so that they may safely use the highest logical unit stresses.

During the middle Eighties the Forth Bridge was built, and there was used in the compression members a high steel of from 76 160 to 82 880 lb. per sq. in., ultimate strength, with a unit stress, for the great circular compression members, 8 and 12 ft. in diameter, of from 15 680 to 16 800 lb. per sq. in., as shown in Table 24. This was in accordance with Continental practice and with the general recommendations of this Committee, but greatly in advance of the units proposed.

Comparing these sensible unit stresses with those which engineers, at the time, were evolving by the laborious use of the Rankine or Gordon formulas, one can at least begin to see the futility of obtaining a great

TABLE 24.—Forth Bridge Stresses, in Long Tons, September 5th, 1884.

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Member.	Dead Load.		Live Load.		Wind Load.		Total.		
	Stress.	Tons per square inch.	Stress.	Tons per square inch.	Stress.	Tons per square inch.	Stress	Tons per square inch.	Pounds per square inch.
Bottom, 12 ft.....	2 282	2.8	1 022	1.2	2 920	3.5	*6 224	7.5	16 800
Top tension.....	2 253	4.4	997	2.0	544	1.1	3 794	7 5	16 800
Vertical, 12 ft....	1 550	3.3	705	1.5	1 024	2.2	+3 279	7.0	15 680
Diagonal struts, 8 ft.....	802	4.1	167	0.8	414	2.1	‡1 383	7.0	15 680
Diagonal ties.....	754	4.6	186	1.2	194	1.2	1 134	7.0	15 680
Horizontal wind Br.....	80	0.9	5	0.1	265	3.0	350	4.0	8 960
Vertical wind Br..	42	0.5	169	2.0	108	1.3	319	3.8	8 512
Center girder, top.....	337	2.4	303	2.2	182	1.4	822	6.0	13 440
Center girder, bottom.....	330	2.3	301	2.1	247	1.8	878	6.2	13 890

* 830 sq. in. † 468 sq. in. ‡ 198 sq. in.

Note.—Same units for both tension and compression.

variety of compressive unit stresses for the various members of the same structure and each one carried out to practically the last pound with ludicrous accuracy. The almost simultaneous proposing of straight-line formulas by Edwin Thacher, M. Am. Soc. C. E., and by Thomas H. Johnson, M. Am. Soc. C. E., caused a great feeling of relief to bridge designers, but never have they entirely departed from a certain amount of hair-splitting accuracy in the design of compression members, so that now one must rejoice because the Committee has failed to promulgate its new formula, $20\,000 - 100\,\frac{l}{r}$, in favor of flat unit stresses, even though one may not agree with these values.

The effort to establish a "Useful Limit Point" does not, in the writer's opinion, tend toward the desideratum, in that one must fix the unit stresses at the highest possible safe value, when finally all the facts are known. Hence, it would seem advisable to fix the U. L. P. at the point where the record first departs from a straight line, or an absolutely safe value to which unit stresses may more closely approach. Then, again, in purchasing steel for compression members, it would seem to be more logical to specify the tests to be made in compression, to give a certain ultimate compressive strength and the corresponding yield point in compression.

Some valuable light might be thrown upon the variation between thick and thin metal, by testing several pairs of columns, from which $\frac{1}{8}$ in. or so of the hard outer skin had been planed, thus proving to what extent the greater percentage of hard skin in the thin sections was responsible for the higher units.

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The tests of columns of tubes, with and without rivet holes, or rather rivets, recently made by Gustav Lindenthal, M. Am. Soc. C. E., might be profitably amplified in order to decide as to the necessity for deducting rivet holes. The attention of designers is called to Mr. Lindenthal's discussion of this Report in which he proves conclusively the necessity for such deduction, and also to the Committee's tacit acknowledgment of this factor of overstress in rivet lines, by its decision to make the rivet spacing in the test columns closer than is usual in practice, to prevent the buckling of the (over-stressed) material between rivet holes. This is set forth in the first paragraph of page 1592.

Further investigation should include a trip of a sub-committee to Europe, to glean all possible facts of value from Continental practice and to learn their methods of producing a more uniform output of steel.

The future programme of tests should cover, not only columns of the usual types with the usual detailing and the various end conditions, but also the eight forms, or test-size columns approximating them, which are probably the best, from a theoretical viewpoint: the old Phœnix; the old Keystone; the Gray column, with proper details; the ordinary box, and the Lindenthal "double box"; the Forth Bridge cellular circular; the Eads Stave Tube; and the writer's cellular octagonal, as described in his discussion* of the paper by O. H. Ammann, M. Am. Soc. C. E., entitled "The Hell Gate Bridge and Approaches of the New York Connecting Railroad over the East River in New York City." These forms are shown in Fig. 7.

The tests made by the engineers of the Forth Bridge are of interest in this particular connection. Mr. Baker

"tested a piece of ordinary stove pipe, 4 in. in diameter and 2 ft. long, made of sheet iron only about a fortieth of an inch in thickness, and found it stood, without buckling, a compressive stress of 15.9 tons per sq. in.; whereas one of the Britannia Bridge rectangular cells, 18 inches square and 8 ft. long, made of plates and angles half an inch thick, crippled under a stress of 13.6 tons, or say 15 per cent. less stress than that sustained by the piece of stove pipe. If flat plates, as much as half an inch in thickness, behave so badly in a rectangular cell only 18 in. square, it is hardly necessary to speculate as to what would be the case in tubes 12 ft. square, which would be the size required for the Forth girders."

The statements recently made, on high authority, that the rapid depletion of the raw materials for steel making would probably result in such abnormally high prices for steel within the next thirty years as practically to prohibit the construction of great bridges, should cause us to consider very carefully any proposed reduction of unit stresses. The column formula of the American Railway Engineering Associa-

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1011.

tion, $16\,000 - 70\,\frac{l}{r}$, between the limits of 28 and 120, with a unit stress of 14 000 lb. per sq. in. below 28, should certainly be adhered to until there are sufficient data to enable engineers to equal or surpass the units used abroad, or as herein shown by the Forth Bridge records. Mr. Fowler.

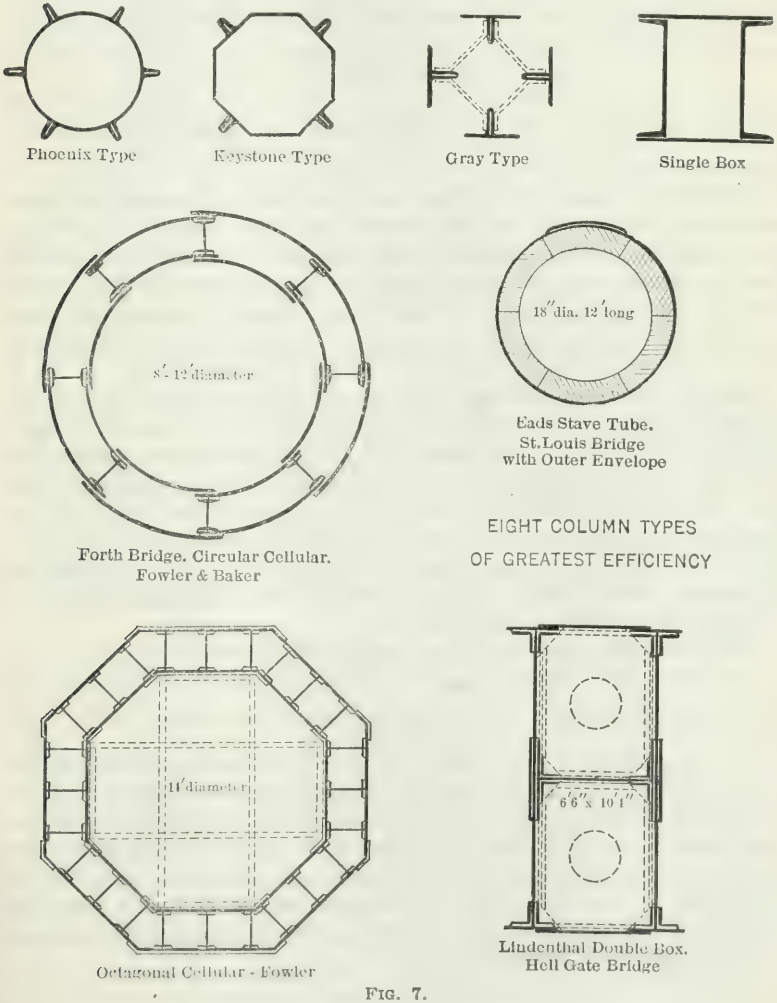

FIG. 7.

Though engineers, for some years, have been preparing their designs on a factor of safety of practically two on the elastic limit, they must, in the writer's opinion, arrive at definite results or limits, both as to stresses and strength of members, and use an even smaller factor of

Mr. Fowler. safety, after having learned the lessons of efficiency which are being forced upon them.

Mr. Waddell. J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—The drastic step taken by the Committee in establishing the intensity of working compressive stress in carbon-steel struts at 12 000 lb. for all values of $\frac{l}{r}$ below 80, and beyond that range reducing it according to the formula,

$$C = 12\,000 - 100\left(\frac{l}{r} - 80\right),$$

up to the limit of $\frac{l}{r} = 120$, does not appear to the writer to be warranted by certain facts.

In the first place, the sections used for testing are small, as compared with those used in bridgework for chord members, although they are more nearly normal for web members. Besides, most of the types of struts tested had open faces, though bridge chords generally have one cover-plate and one laced face. Again, there should have been a series of tests for comparatively small values of $\frac{l}{r}$, because it is hardly conceivable that a short block will not resist compression at least as well as any member made of the same metal will resist tension. In the years long gone by, it was the prevalent opinion that such short struts could be stressed somewhat higher per square inch than the tension members; then custom decreed that the limiting intensities of working stresses should be alike; but now comes a further and most radical cut, amounting to 25 per cent. The Engineering Profession should pause a little before incorporating this innovation into its standard bridge specifications.

Some 14 years ago the writer made twelve tests of struts of both carbon steel and nickel steel, the sectional area thereof being 17.5 sq. in., and the lengths 10 and 30 ft., making the values of $\frac{l}{r} = 27$ and 81, respectively. The results of these tests were published by the Society in 1909 in a paper entitled "Nickel Steel for Bridges",† from which paper it can be seen that the carbon-steel struts gave, as averages, unit elastic limits of 28 800 lb. for the short ones and 16 800 lb. for the long ones, the corresponding unit ultimate strengths being 39 200 and 30 500 lb. It is true that all the struts had hinged ends, hence, to compare the results with those found by the Committee, they should be increased about in the ratio of

$$R = \frac{16\,000 - 60\,\frac{l}{r}}{16\,000 - 80\,\frac{l}{r}}.$$

* New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 101.

Substituting $\frac{l}{r} = 27$ and $\frac{l}{r} = 81$, we have, respectively, 1.04 and 1.17. Mr. Waddell.

Applying these figures, we find, for the elastic limits, 29 950 and 19 660, and, for the ultimate strengths, 40 770 and 35 690.

It is difficult to compare these figures with the averages of those found by the Committee, because it did not determine the elastic limit, but, instead, a new function designated "Useful Limit Point". It is true that the writer's method of determining the elastic limit was arbitrary; but it was conservative. For the value of $\frac{l}{r} = 85$ (which, for comparison, is sufficiently close to the writer's superior limit of 81) the Committee found, as averages, for the "Useful Limit Point" and the ultimate strength, respectively, 27 100 and 30 200. The former possibly might properly be compared with the average of the results obtained by the writer, in the case of the longer columns, for a permanent set of 0.015 in., *viz.*, 21 300, which, of course, as before, must be multiplied by 1.17, making 24 920.

Comparing the ultimate strengths, we have 30 200 lb. per sq. in. found by the Committee, against 35 690 lb. found by the writer. As the quality of the metal was practically the same in both cases, this great difference is presumably due primarily to the fact that the writer's struts were detailed so perfectly that they developed the greatest possible strength from the section, which apparently was not the case in some of the Committee's test struts.

The writer's latest specifications for designing* permit the following intensities of working stresses for carbon-steel columns:

For fixed ends,

$$C = 16\,000 - 60\,\frac{l}{r},$$

and for hinged ends,

$$C = 16\,000 - 80\,\frac{l}{r}.$$

Substituting in the latter 27 and 81 for $\frac{l}{r}$, we have, respectively, 13 840 and 9 520. Comparing these figures with the results of the writer's tests, which gave for the elastic limits 28 800 and 16 800, we find the ratios to be 0.48 and 0.57. As the aim in preparing the formulas was to make the intensity of working stress about one-half of the elastic limit, it is evident that there is an error of 2% on the side of safety for the short columns, and one of 7% on the side of danger for the long columns. It must be remembered, though, that for the latter the writer used a very conservative method in trying to locate the elastic limit.

* "Bridge Engineering", Chapter LXXVIII.

Mr. Waddell.

These experiments by the writer show conclusively the fallacy of using a constant intensity of working stress up to a limit of $\frac{l}{r} = 80$; because (when corrected for fixedness of ends) the elastic limits are 29 950 for $\frac{l}{r} = 27$ and 19 660 for $\frac{l}{r} = 81$. Taking one-half of each of these, we have, for the allowable working intensities, approximately 15 000 and 10 000 lb., though the Committee would use 12 000 lb. for both cases. This shows that the Committee's assumption is illogical. The writer's formula for fixed ends makes these intensities 14 380 and 11 140. Therefore, it does not seem justifiable to stress as high as 12 000 lb. for $\frac{l}{r} = 80$; and it is decidedly uneconomic to use that intensity for $\frac{l}{r} = 20$ or 30, as would be the case in chord sections of many railroad bridges.

In the writer's 42 years' experience, connected mainly with bridges, he cannot recall a single instance of the failure of a properly designed compression member, but he has known of tension members failing, although, truth to tell, not many of them, except in detail. Were it true that we were over-stressing short columns some 35% when using for fixed ends the old standard formula,

$$C = 18\,000 - 70\,\frac{l}{r},$$

there inevitably would have been numerous column failures in old bridges in which the actual intensities of working stress on the tension members approached the danger limit.

As for the great differences in resistance between struts fabricated with thin metal and those fabricated with thick metal, the adoption of more thorough and cooler rolling for the latter would probably reduce that difference materially. In any case, though, we should certainly need more experiments on this point before upsetting fundamentally the standard practice of American bridge engineers.

Mr. Godfrey.

Edward Godfrey,* M. Am. Soc. C. E. (by letter).—The Final Report of this Committee contains some very valuable data, and deserves the study of all structural engineers. It has long been evident to thoughtful engineers that columns, as commonly designed, are far from having the alleged factor of safety of four which they are so commonly said to have. It has also been evident that there is no warrant in fact for the unit stress, used in tension members, to be applied also in compression members, even though there is a reduction for slenderness.

* Pittsburgh, Pa.

Mr. Godfrey.

On the face of the results of these tests one striking fact is the small difference between the values for the strength of columns of 50 and 120 radii in length. At first, this would appear to upset formulas and ideas on the relative strengths of short and slender columns. However, correct interpretation of the results brings these tests into harmony with both theory and practice. In other words, correct column theory is not proved to be unreliable, and good practice is not proved to be unsafe; though the fact is that, for indiscriminate designing, a lower unit ought to be used in the column formula.

The three ratios of slenderness used for most of these tests are 50, 85, and 120; but it is not pointed out that these ratios contemplate lengths which are between perfectly fixed ends. It is true that the column formula in general use in America—the straight-line formula—is applied without any reference to the rigidity of the ends; but it is also true that examples of truly fixed-ended columns are so rare as to be considered negligible. Columns which butt squarely on a masonry base are fixed-ended, and correspond, at one end only, to the columns of these tests. Practically all other compression members of any size in structures may be considered either continuous or pin-ended at the ends, but few are really fixed-ended.

The real length of these test columns, considered as pin-ended, that is translated into practice, is just half of that shown in the Committee's report. The ratios of slenderness are then 25, 42.5, and 60, instead of 50, 85, and 120. The significance of this distinction is seen when a line is plotted representing the ordinary straight-line formula and is compared with the Committee's results of tests after correcting them for slenderness ratio.

The writer has deduced* a purely theoretical column formula which, although it is very complex, gives a locus which is almost identical with the straight-line formula in the range of practical column ratios as used in structural work. The one assumption on which the derivation of this formula is based is that the column is imperfect, and that the imperfection exhibits itself in the deviation of the column from a straight line by an amount of $\frac{1}{500}$ of the length. The formula shows the direct load per square inch which will give an extreme fiber stress on the assumed imperfect column of 16 000 lb. per sq. in.; and this load is found to agree, almost exactly, with $16\,000 - 70\,\frac{l}{r}$ up to the ratios of slender columns.

The average elastic limit of short pieces in compression is not far from 32 000 lb. per sq. in., as shown by the Committee's report. If now, the values of the straight-line formula or the writer's formula are doubled (on the assumption that the factor of safety is two), and if the line is plotted on the Committee's Plate XXXVI, correcting the

* In his book, "Steel Designing."

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ratios of slenderness to just half of those shown, the line will traverse right through the dense portion of that plate, where the greater number of test results lie. Furthermore, the shape of this line agrees pretty closely with the shape of the various lines for the individual column groups. This would not be the case with the false ratios given on Plate XXXVI.

Engineers have long recognized that when the elastic strength of a column has been reached, or has been exceeded by a small amount, it is only a matter of time or the repetition of the same load, until the column fails. The same is true of tension members, though the time and the repetition of stress need to be greater. Chains are commonly used for loads, which give fiber stresses that exceed the elastic limit of the steel; and they commonly fail under the treatment, unless they are annealed in order to restore the weakened metal.

These tests do not give surprising results, nor results out of harmony with theory, when that theory recognizes that a column is an imperfect member and one in which the imperfection intensifies the destructive effect of loading near the elastic limit. A tension member which is bowed will be partly straightened out under high load, and thus the effect of this imperfection, in adding bending strains, is diminished.

The real factor of safety of a tension member subject to a stress of 16 000 lb. per sq. in. is not much greater than two, for the real elastic limit of ordinary structural steel is not much greater than 32 000 lb. per sq. in. The real factor of safety of a compression member having a stress of $16\,000 - 70\,\frac{l}{r}$ per square inch would be about two, with proper design and metal which is properly rolled, that is, worked down enough to refine and harden it. Because of uncertainties of design and of the quality of the material, however, the writer believes that a value lower than 16 000 should be used in the straight-line formula. He believes that $15\,000 - 65\,\frac{l}{r}$ would be a good formula for compression members.

The writer has no apprehension as to the safety of columns designed by the ordinary straight-line formula, when the structures are properly designed, and when they are not overloaded, for he believes that a factor of safety of two is sufficient when the ultimate limit of strength on which this factor is based is an elastic ultimate, and not an ultimate found by speedy loading and rupturing of a member which would fail under much less load if given time.

The Euler load, with a factor of safety of five, is used by European designers. This is an absurdity, for the reason that where the Euler formula has practical application, namely, in slender columns, clearly

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outside of the range of good structural design, no factor of safety of five is needed. There are long wooden derrick booms, being used every day, which have a factor of safety of less than two, based on the Euler formula. The use of the Euler formula for short columns is worse than a guess. At 60 radii that formula shows an ultimate unit stress of 80 000 lb. The Committee's tests show 19 200 to 31 100. At 25 radii the Euler formula shows an ultimate strength of 460 000 lb. per sq. in. No comparison is needed. What meaning could a factor of safety of five have where the ultimate strength shown in the formula bears no relation whatever to the real ultimate strength of the column? The saving feature of European specifications is the upper limit, which, the writer believes, is about 14 000 lb. per sq. in.

Cause for apprehension, backed by the results of these tests, lies in common methods of design, which frequently ignore eccentric strains, and in the overloading of structures.

The report* of the Committee on Iron and Steel Structures of the American Railway Engineering Association, makes certain recommendations. The report states:

"Unit strains in tension to the extent of 26 000 lb. in structural open-hearth steel and 22 000 lb. in wrought iron will not, in themselves, be sufficient justification for suspending traffic or condemning the structures."

Again it is stated:

"That means if you cut down speeds you can run heavier overloads. It would mean practically taking a 60-ft. girder span at full speed on an overload of about 71 per cent., while at slow speed the overload would be something like 122 per cent."

These ideas are again set forth in Vol. 15 of the *Proceedings* of that Association. It is further stated that "we know from numerous tests and long experience" that properly designed bridges in good condition will safely withstand an overload of 50% without any traffic or speed regulation, and, if speed is regulated, an occasional overload of 100 per cent. Further:

"An E-50 A. R. E. Assoc. Specification bridge is a good and economical type with sufficient strength to carry safely, in regular unrestricted service, the heaviest locomotives that can be safely operated without a possible complete revision of present standard clearances."

Now, the relative impact between low speeds and high speeds is something that nobody knows anything about. Some tests have shown less impact for a high speed than a lower one. Impact is caused by roughness of track and lack of balance of driving wheels. If the compression members of a bridge were loaded from 50 to 100% above the ordinary units, where were the factors of safety?

* *Proceedings*, Am. Ry. Eng. Assoc., Vol. 9.

Mr. Burr. WILLIAM H. BURR,* M. AM. SOC. C. E. (by letter).—The work of the Special Committee on Steel Columns and Struts has extended over a period of 9 years, one progress report having been made about a year after the creation of the Committee. After such a long period of activity, it is only reasonable to suppose that the Committee would have covered much ground in a thorough and conclusive manner, especially in view of the fact that a great testing machine of unusual power and accuracy had been placed at its disposal. It is a little disappointing, therefore, to find that the most used type of cross-section of steel column up to the present time is not included in the scope of the Committee's consideration. These tests have not included any column section in which lattice bars or stay-plates are used for holding the component parts of the column in place so as to make an effective compression member.

It is obviously of real importance to learn by actual test the resisting capacities of columns with spacing details, and also of those which are built up or rolled so that they do not require such spacing details as lattice bars and spacing plates or battens; but, as those of the former class are far more numerous than those of the latter, up to the present time, the work of the Committee would have been much more nearly complete if the investigations had included both classes. Certainly, the element of time was not lacking to accomplish such a result.

All the columns were tested with "flat ends", unquestionably the most uncertain end condition which could be seriously considered for such a purpose. Clearly, if the end bearings could be made mathematically exact and at right angles to the axes of the columns tested, there would be reached the condition of true fixedness, at least at the beginning of the test of each column.

That condition, however, would immediately disappear, and the column would at once become eccentrically loaded—with constantly increasing eccentricity—as the load on the column increased. It will presently be shown that an increasing eccentricity of loading took place in probably every column tested in the entire series. In other words, the series of tests is one of columns eccentrically loaded, and with an eccentricity constantly increasing, and rapidly so after passing the limit of elasticity.

It is essentially impossible to face the ends of a column so as to make a strictly accurate fit at each end. Apparently, this was the case in this investigation, as the Committee states:

"Considerable difficulty has been found in securing bearing blocks of sufficient accuracy, and it was necessary for the Bureau to go over all the blocks and dress them to a surface with a variation of not more than plus or minus 0.001 in."

* New York City.

Mr. Burr.

Again, in the instructions issued by the Committee for testing, there is found the statement: "The ends of the columns shall be dressed flat to reduce the amount of shimming to a minimum." It can scarcely be expected that the shimmed flat end of a column would be in condition to be uniformly compressed, however carefully the shimming might have been done. In fact, it would scarcely be too much to state that any shimming whatever would be practically certain to render centric loading of the column impossible, and the extent of the eccentricity induced would be quite impossible to determine. The report does not contain any satisfactory statement as to the reason for the choice of this uncertain flat-end condition for the investigation. It was recognized among the older investigators that the flat-end condition introduces variables of eccentric bearing at one end which differ from those at the other, with the attendant eccentricity of loading, which cannot be estimated, carrying with it a corresponding amount of extraneous and uncertain bending in the column. This variable eccentricity and bending moment reduces obviously the resisting capacity of a column by an amount quite impossible to determine.

Furthermore, the flat-end column, with a few rare exceptions, is now never used in structures in other shape than columns of cast iron, and columns of this material have nearly disappeared in first-class work. Even in heavy riveted structural joints, the endeavor is essentially universal to make all lines of stress intersect in a point which will insure centric loading; but, even there, the elastic motions of the intersecting members prevent the fixed-end condition. It is for this and other substantial reasons that some engineers have well contended, as the writer believes, that the pin end corresponds more nearly than any other to the general conditions under which structural columns are used; and, hence, that the results of column tests with pin ends may be used safely and properly for practically all structural columns wherever they may be found.

Fortunately, the measurements of the strains at different parts of loaded columns when under test enable the preceding observations to be rather markedly illuminated. The report states that these strains were measured to $\frac{1}{5000}$ in. As such measurements, to be of greatest value, must be as accurate as possible, it is a matter of surprise that a more accurate measuring apparatus was not used, as it is now quite possible to measure, by the aid of a perfect screw, to $\frac{1}{50000}$ in., and by close estimation to less than 0.00001 in. A geared dial is not the most accurate instrument available at the present time, but it may be accepted for the present purpose.

Prior to using the data in the tabular results of the report, it is proper to state that, with this important and rather analytic subject before a committee of engineers for the period of 9 years, it would be reasonable to expect at least the fundamentals of a thorough

Mr. Burr. analytic statement of the typical or principal stresses in a centrically-loaded column, if not supplemented by at least a simple outline treatment for eccentrically-loaded columns. As a matter of fact, there is not in the entire report any matter whatever, either analytic or other, bearing on the stresses found at various points of the cross-section of a loaded column, either at the ends, or at the center, or at any other section. The only intensity of stress given anywhere is the mean or average intensity, which, of course, is the same from one end of a column to the other. All conclusions reached, so far as they depend on any intensity of stress, are expressed in terms of that mean or average. Nor has there apparently been any effort on the part of the Committee to interpret in any way the significance of the varying strains at different parts of the column, although some highly important conclusions may be based on results thus disclosed.

It is sufficient here to observe that, in general, the state of stress in any column, as has long and well been known, is a combination of a uniform stress and a stress couple, as shown in Fig. 8. In that figure, $D\ K = C\ L$ is the uniform intensity of stress on any section, $K\ L$, and is the mean or average intensity shown as "Unit Ultimate Strength", or "Average Ultimate Strength", in Tables 1, 2, etc., of the report. As a matter of fact, however, the actual intensity or compressive stress which determines the failure of the column is the quantity, $H\ L$, of Fig. 8, and the comparatively small intensity or compressive stress, $F\ K$, is found on the opposite side of the section. It can readily be seen from Fig. 8 that, comparatively speaking, the intensity of stress, $C\ L$, may not be of much consequence, inasmuch as the maximum intensity, $H\ L$, actually exists at the same point.

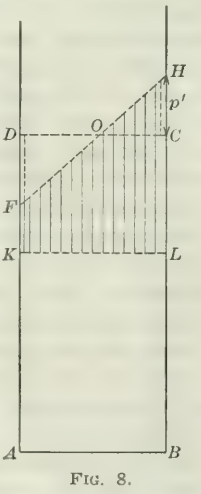


Fig. 8.

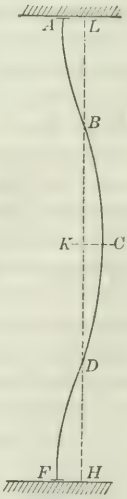


Fig. 9.

Referring to Fig. 9, the full line, $A\ C\ F$, represents the center line of a column with flat ends at A and F, the deflections being much exaggerated. If the ends are accurately flat and at right angles to the axis of the column when unloaded, and if the bearing surfaces are rigidly unyielding, the imposed load, if not too great, will cause the center line of the column to take the reversed curve shown, $B\ L$ being equal to $B\ K$, and similarly for the other half of the column. The deflection, $A\ L$, will also be equal to $K\ C$. Hence, the center deflec-

tion, $K\,C$, will be equal to the eccentricity of loading, $A\,L$. The maximum bending moment in the column, at C and at A, will be equal to the column load multiplied by $K\,C$, or the eccentricity, $A\,L$. As a matter of fact, the unavoidable inaccuracies of flat ends and the elastic motions caused by column loading derange to some extent the conditions outlined, although the principal features are, in general, approximately maintained. Bad fitting of the ends, or other influences of a similar kind, may even bring the point of reversed curvature, B, up near to L (Fig. 9), thus approximating the condition of a round end. In the present instance, however, the end fitting was doubtless unusually accurate, producing closely the ideal conditions described. With the increasing load, however, the elastic motion at the ends of a column would cause increased eccentricity, as is actually shown by the increasing measured strains. Mr. Burr.

These matters may be illustrated by taking the data for Test No. 30 first with a total load of 113 600 lb. and then a total load of 284 000 lb. The compressions for the gauged length of 8 in. at the middle of the column are 0.0026 and 0.0026 in., for Points 1 and 3, and 0.0025 and 0.0028 in. for Points 5 and 7; which shows that there was irregularity of deflection, but that there was a vertical deflection. Using 30 000 000 for the modulus of elasticity, with the preceding compressions for 8 in. in length, the intensities of stress are found to be:

For Point 1..... 9 750 lb. per sq. in.
" " 3..... 9 750 " " " "
" " 5..... 9 375 " " " "
" " 7.....10 500 " " " "

These intensities show that the column deflected upward with the load of 113 600 lb., giving the following mean fiber stresses:

At Points 1 and 7.....10 125 lb. per sq. in.
" " 3 " 5..... 9 563 " " " "

562

Hence, in Fig. 8, $p^1 = \frac{562}{2} = 281$ lb. per sq. in.

As the moment of inertia of the cross-section of this column is $I = 64.54$, and as the half depth of the column is 3 in., the moment of the stress couple, indicated in Fig. 8, is

$$M = \frac{p^1 I}{d} = \frac{281 \times 64.54}{3} = 6\,045 \text{ in-lb.}$$

Hence, the eccentricity of the load was $\frac{6\,045}{113\,600} = -0.053$ in.

The negative sign is used to indicate upward deflection.

Mr. Burr. When the load was increased to 284 000 lb., the strains in the gauged 8-in. length were:

At Point 1, 0.0068 in.; hence the intensity of stress = 25 500 lb. per sq. in.
At Point 3, 0.0086 in.; hence the intensity of stress = 32 250 lb. per sq. in.
At Point 5, 0.0078 in.; hence the intensity of stress = 29 250 lb. per sq. in.
At Point 7, 0.0071 in.; hence the intensity of stress = 26 625 lb. per sq. in.

Mean fiber stress at Points 1 and 7....26 063 lb. per. sq. in.
" " " " " 3 " 5....30 750 " " " "

These intensities show that the deflection was downward. Using the foregoing data, the moment of the stress couple is

$$M = \frac{2\,344 \times 64.54}{3} = 50\,419 \text{ in-lb.}$$

The eccentricity of the load was $\frac{50\,419}{284\,000} = +0.18$ in.

Again, the recorded results of Test No. 54, Type 2, may be used in the same way. With a load of 96 600 lb., the strains on a gauged length of 8 in. at mid-length of the column are:

At Point 1, 0.0025 in.; hence the intensity of stress = 9 375 lb. per sq. in.
At Point 3, 0.0023 in.; hence the intensity of stress = 8 625 lb. per sq. in.
At Point 5, 0.0026 in.; hence the intensity of stress = 9 750 lb. per sq. in.
At Point 7, 0.0027 in.; hence the intensity of stress = 10 125 lb. per sq. in.

These results indicate that the deflection is horizontal and to the "south". Taking half the difference of the mean intensities at Points 1 and 3, and 5 and 7:

$$p^1 = \frac{938}{2} = 469 \text{ lb. per sq. in. (Fig. 8).}$$

The moment of inertia of the cross-section is 53.84, the width of the section being 8 in. The moment of the stress couple is then

$$M = \frac{469 \times 53.84}{4} = 6\,313 \text{ in-lb.}$$

The eccentricity of the load, therefore, was $\frac{6\,313}{96\,600} = +0.065$ in.

Finally, with a load of 280 140 lb., the corresponding strains in the gauged length of 8 in. were: Mr. Burr.

At Point 1, 0.0068 in.; hence the intensity of stress = 25 500 lb. per sq. in.
At Point 3, 0.0060 in.; hence the intensity of stress = 22 500 lb. per sq. in.
At Point 5, 0.0105 in.; hence the intensity of stress = 39 375 lb. per sq. in.
At Point 7, 0.0116 in.; hence the intensity of stress = 43 500 lb. per sq. in.

Using half the difference of the two mean intensities at Points 1 and 3, and 5 and 7:

$$p^1 = \frac{17\,438}{2} = 8\,719 \text{ lb. per sq. in.}$$

Hence, the moment, M, of the stress couple is

$$M = 8\,719\,\frac{53.84}{4} = 117\,358 \text{ in-lb.}$$

The eccentricity, then, is $\frac{117\,358}{280\,140} = +\,0.42$ in.

These computations show that the eccentricity of loading increases with the increase of load, and that, even within the elastic limit, in Test No. 54, it was more than 5% of the width of the section. After the elastic limit is passed, the eccentricity increases rapidly, and increases the moment of the stress couple with equal rapidity. This is the explanation of the hastening failure of the column after passing the elastic limit.

The intensities of stress obtained by actual strain measurements demonstrate in a marked manner the gross error likely to be committed in basing analytic conclusions on the mean or average intensity of stress in the cross-section of a loaded column. In the case of Test No. 30, with a total load of 284 000 lb., and with a mean intensity of stress of 25 000 lb. per sq. in. at the mid-length section of the column, the strain measurements show that the mean maximum compression in the same section is 30 750 lb. per sq. in., which is 20% more than the average. The results found in connection with Test No. 54 are still more striking with a total load of 280 140 lb., giving a mean intensity of 29 000 lb. per sq. in. on the mid-length section. The strain measurements disclose a mean maximum intensity of compression of 41 400 lb. per sq. in., an increase of nearly 50% above the mean intensity. This column gave an ultimate resisting capacity of about 30 000 lb. per sq. in. as an average intensity over the entire section, but the strain measurements show that the maximum intensity of compres-

Mr. Burr. sion in the section might have been as high as from 45 000 to 50 000 lb. per sq. in., or even higher.

A large but undeterminable part of this excess of the maximum intensity of compression over the mean is directly due to the eccentricity of the column loading. Even if it is admitted that the strain measurements were not as highly accurate as could be desired—which is probably true—this analysis demonstrates conclusively that the varying and rapidly increasing eccentricity of loading in these tests is far too large an element to be ignored, and that any part of the investigation bearing on, or influenced by, the intensities of compressive stress in the cross-section of the column must take into account, at every step of the operation, the maximum intensity of stress, and not the mean.

The report contains the details of stress measurements of Tests No. 30 and 54 only. If similar details of other tests were printed, doubtless more marked cases even than No. 54 would be found.

The Committee emphasizes the fact that the programme of investigation was arranged so as to eliminate as far as practicable "all variables except that of form". The meaning of this statement is not quite clear. It certainly could not have been the intention of the Committee to eliminate the ratio, $\frac{l}{r}$, for it has been given a variation from 50 to 120, and value would have been added to the results of the tests if other ratios than the three selected had been used. Again, it has been seen that the eccentricity of loading involved was highly variable. The carbon content was also varied. Unfortunately for the designers of structural steel columns, as has already been observed, it was decided to eliminate from the columns tested such spacing pieces as lattice bars, tie-plates, and similar details. Details of this kind have been used so widely in the past, and are still likely to be much used, that full-size tests determining their value and dimensions would be highly important in steel structural design, even at the present time.

It has been a fundamental principle in the best design of cross-section for a steel column to distribute the metal so as to make each of the different parts individually self-supporting, as far as practicable, and mutually supporting as parts of a whole, so as to maintain the full value of the radius of gyration in the direction of bending, up to the instant of actual failure. Convenience of construction, accessibility for painting, and other matters of a similar nature do not permit a perfect application of such fundamental considerations, but they do influence design strongly. The Phœnix column probably fulfilled these ideal conditions more nearly than any other section ever devised, but the inaccessibility of the interior of the column, and

difficulties in detailing, would prevent any such section from coming into general structural use. Mr. Burr.

The value of this investigation would have been much enhanced if its scope had been sufficiently increased to throw the light of actual test results on the relative value of a number of forms of cross-section which, in the past, have been much used, and are still. The type of section used almost entirely by the Committee, namely, with the individual projecting flat and unsupported parts, is the least effective of all for purely column purposes, although, in one shape or another, it is widely used, and the results are correspondingly valuable.

The data obtained by the tests and shown in the tables of the report are highly interesting. The long and well-known decrease of carrying capacity due to heavy sections—that is, those of thick metal—stands out prominently, although in some instances it is masked by the varying content of carbon, as the Committee allowed "the mill to change the carbon content so as to endeavor to produce the same strength for the light material as for the heavy". This attempt to cure the influence of lack of work between the rolls by chasing it with a varying carbon content proved ineffective, as was to be expected. By retaining the same carbon content, a well-determined effect of insufficient work would have been obtained.

It is greatly to be regretted that the report did not include a graphical representation of the ultimate resistances, so as to show at a glance the actual relation existing between them and the column ratio, $\frac{l}{r}$. It is true that a more satisfactory probable curve could be drawn with additional results for the intermediate ratios, 68 and 138, as well as for the smaller value, 35. On Fig. 10 are shown the three main groups of tests, as well as the smaller group for which $\frac{l}{r} = 155$, and the single test for which that ratio is 20. Each of these groups actually shows considerable variation, but not more than is to be expected under the condition of flat ends, with the varying eccentricity of the increasing loading, the added influence of thickness of section, and the varying content of carbon. The extra heavy section of Type *5B* gives strikingly low ultimate resistances, lower indeed than one would expect, even with as much thickness of metal as $1\frac{1}{4}$ in. The content of carbon may have been an important element in this case, but it does not appear to be given in the report. In any event, however, no experienced engineer would permit as high working resistances in columns having excessively thick parts as in those of lighter section.

The value of this graphical representation is that it shows at a glance, among other things, that the working stresses prescribed for columns with end conditions such as those used in this investigation

Mr. Burr. should vary reasonably, not only with the column ratio, but also with other elements, such as the carbon content and the thickness of metal in the different parts of the column section, although too much importance should not be attached to this latter feature unless the thickness is sufficient to decrease markedly the work done between the rolls. If the carbon content is to be varied so as to cure insufficiency of work in rolling, careful tests should be made to determine the desired content with reasonable accuracy, so as to bring the resisting capacity of the column up to the required amount. In every feasible case, however, the needed amount of work should be prescribed and secured.

In order to make a comparison with some of the best of the older tests, there are shown on Fig. 10 the results of the tests of medium-steel angles with flat ends made by the late James Christie,* M. Am. Soc. C. E. It would have added much, both to the interest and value of this report, if the older tests of wrought-iron and steel columns had been shown in comparison with those of the present steel flat-end columns, wherever they were made by reputable experimenters. It is a mistake to assume that the best of those older tests were not sufficiently accurate to be compared with these modern results. Although it is true that the older machines were not as extremely accurate in some respects as the machine used in this series of tests, in some other respects they were at no disadvantage, and the results obtained may be accepted as essentially correct. Indeed, the tests of flat-end angles made by Mr. Christie, taken in connection with others made by him at the same time, probably constitute the most interesting, and in some respects the most valuable, of all structural tests that have ever been made. His papers constitute an almost unrecognized classic in the literature of testing materials. He was, in the best sense of the word, a perfect mechanic. His intuitive sense of the fitness of details and character of appliances to secure desired—and in many cases delicate—results has seldom been equalled by any experimenter in structural materials, and his interpretation of results showed him to be, not only an engineer, but an engineer physicist.

It will be noticed that Mr. Christie's unit results are materially higher than those obtained by the Committee, up to the column ratio of 120, and practically the same at the column ratio of 155. This is doubtless largely due to the fact that he experimented with single angles, and the Committee with built-up sections. With the lower values of $\frac{l}{r}$, the riveting of a built-up section is an element of weakness, as the parts are not held rigidly in position in relation to each other, and, consequently, a slight though quite imperceptible relative motion will correspondingly reduce the unit carrying capacity. It is

* *Transactions*, Am. Soc. C. E., Vol. XIII (1884), pp. 253 and 285.

also partly due to the fact that more accurate flat-end fitting can be made with the smaller single-member angle struts than with the full-size columns used by the Committee. The weakening effect of the increasing eccentricity with the increasing load in the latter would be relatively greater than in the case of the single angles. Mr. Burr.

The angles tested by Mr. Christie were of mild steel, with a carbon content of 0.11 to 0.15, which is distinctly less than that found in the material used by the Committee. It may be properly observed, of course, that results obtained from the small sections of single angles are scarcely comparable with those from full-size columns. Although that is true, the comparison is legitimate, however, for it is at least illuminating as to the general features of the results.

In the older wrought-iron and steel column tests such graphical representations were always accurately made and carefully studied, enhancing greatly the value of those investigations and forming the only reliable basis for the convenient determination of the empirical quantities used both in Gordon's and the straight-line formulas. In the present instance such a graphical representation of results gives a much clearer and more complete view of the different resisting capacities for light and extremely heavy sections than is possible by any tabular statement; and other features are brought out with similar definition. A mean-resistance curve may be drawn through the mean points of the stress groups to represent the straight-line formula within the limits of the tests, or two such mean-resistance lines may be drawn, one through the mean points for the light sections and another through the mean points for the heavy sections.

In scrutinizing such an important report, it is natural to look for evidences of material advancement in the knowledge of the subject investigated. The principal, if not the only markedly original, feature disclosed in the report is what the Committee calls the "Useful Limit Point". When a new scientific term is introduced into any field of research, it generally indicates some important physical feature or discovery corresponding to some new state or quality of the material under test, like the elastic limit or yield point in the field of testing materials. The "Useful Limit Point", however, is not a term corresponding to or signifying any new feature or quality of the material at any particular intensity of loading; in fact, it does not indicate anything whatever regarding the condition of the material in the column under test, except that it must be "somewhere" above the yield point. It is simply an arbitrary term applied to an arbitrarily chosen point on the envelope of the stress-strain curves of the column. It is a point determined by a tangent having a completely arbitrarily chosen inclination, that is, half that of "the last run-up line."*

* The literary elegance of some modern scientific terms or definitions is indeed striking.

Mr. Burr. It is unfortunate, to say the least, that the Committee should have introduced a superfluous term where simplicity is needed. It is difficult to realize why this specially arbitrary and substantially meaningless term can be supposed to acquire such value as the report accords it. It varies greatly, as does the yield point and ultimate resistance, as indeed it is bound to by the nature of the case. It has to be established by the construction of an envelope for stress-strain curves, both above the yield point and below the elastic limit. It has no definite relation to either the yield point or the ultimate resistance. In fact, it is not an exaggeration to say that it has no physical value whatever. As a matter of fact, the yield point may be located just as accurately, if not more so, and has the material advantage of meaning something in the column test.

A feature which will probably challenge some criticism is that part of the report in which is considered the safe working stress for columns "conservative enough to cover the wide range in the results shown by the Committee's programme." Inasmuch as "the usual working stress in tension is approximately one-half the elastic value of the metal", the Committee arbitrarily assumes that "columns should have a safety factor of at least two, based on the U. L. P., in order to be on a parity with tension members." It certainly is not entirely clear why a safety factor of two, based on "the elastic value of the metal" in tension, is on a parity with the safety factor of two in columns based on the Useful Limit Point. Assuming that "the elastic value of the metal" means the elastic limit, there is no conclusive reason whatever for assuming that one-half the elastic limit in tension gives the same factor of safety as one-half the Useful Limit Point of a column in compression. In fact, the two things are radically different, when considered either in reference to the degree of safety in loading or as to the physical condition of the material.

The Committee then takes the average of all the Useful Limit Points of the column tests for the column ratios, 50 and 85. It then takes a mean between that average and the lowest observed value, which gives 23 450 lb. per sq. in. This value is then called 24 000 lb., and the factor of two is applied to it, giving 12 000 lb. per sq. in. for a sort of unit or base working value, which the Committee recommends, it being assumed that static loads only have been considered, "and that a percentage of the static stress will be added to cover the impact due to moving loads."

The averaging of the aggregate test results, for the column ratios of 50 and 85, without regard to the variation of that ratio, and then the averaging of that average with the lowest result obtained with excessive thickness of metal is so crude and irrational as to be entirely

Mr. Burr.

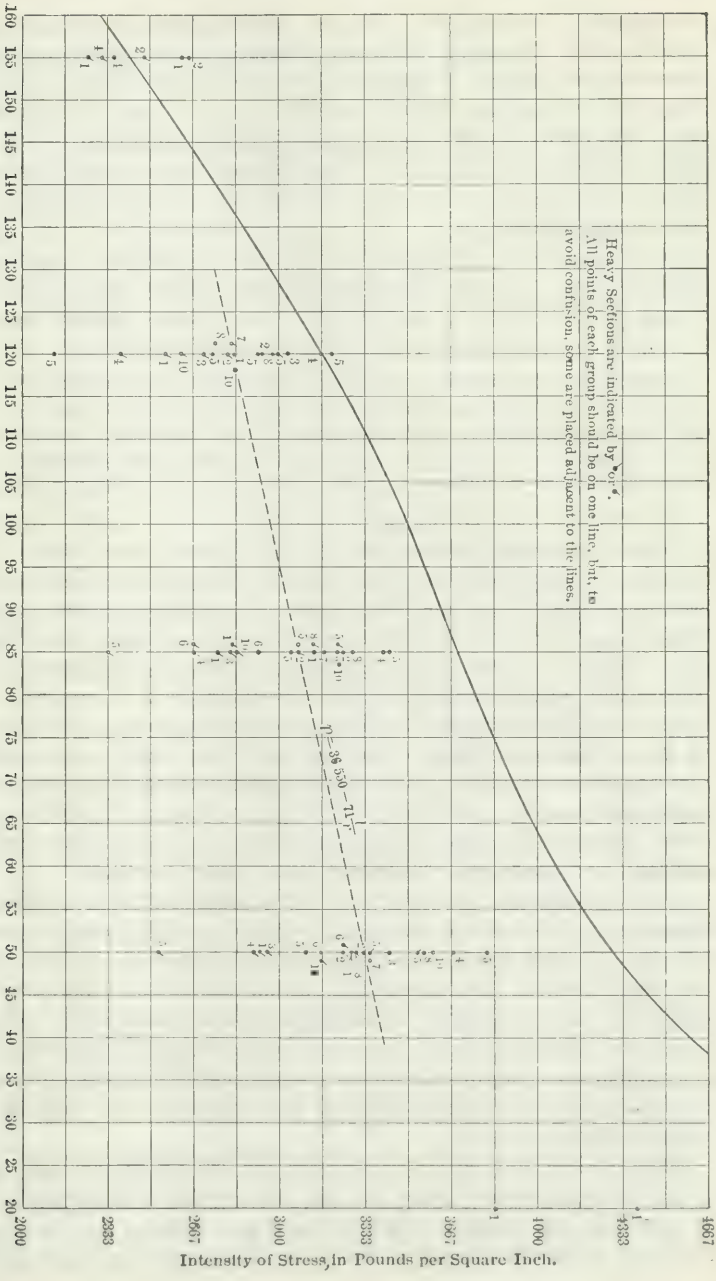

Heavy Sections are indicated by ●.
All points of each group should be on one line, but, to avoid confusion, some are placed adjacent to the lines.
$\eta = 38{,}550 - 71\frac{1}{7}\frac{l}{r}$
Values of $\frac{l}{r}$
160 155 150 145 140 135 130 125 120 115 110 105 100 95 90 85 80 75 70 65 60 55 50 45 40 35 30 25 20
2000 2333 2667 3000 3333 3667 1000 4333 4667
Intensity of Stress, in Pounds per Square Inch.

FIG. 10.

Mr. Burr. indefensible, irrespective of the use of the arbitrary "Useful Limit Point". Fig. 10 shows that the decrease of carrying capacity, in increasing the column ratio from 50 to 85, is too material to be neglected. Furthermore, the test results for the Type 5*B* section are seen to be phenomenally low, and should not be included in the determination of a general formula or resistance curve. Any experienced engineer would make due allowance in his working stress for such extra heavy sections.

It is greatly to be regretted that the Committee takes such a hopeless view as to the practicability of determining a reasonable column formula from the tests of full-size columns. In the paragraph at the bottom of page 1630 occurs the following statement:

"Numerous theoretical formulas have been promulgated from time to time, all of them based on entirely elastic material of uniform grade. At the present time it seems useless to the Committee to attempt to write a rational column formula, from a theoretical standpoint, with the possibility of variations of 28% in the strength of the material."

It is true that enthusiastic analysts have proposed from time to time some rather extreme and usually extremely complicated column formulas, but probably few if any experienced engineers have regarded them seriously. The only two types of formulas which have been widely used are the so-called Gordon's and straight-line formulas. It is certainly a fact that Gordon's formula is established from a "theoretical standpoint", but that is equally true of probably nine-tenths of all the practical formulas used by engineers. It is true of all beam formulas, which, like Gordon's, are fitted with empirical quantities or coefficients giving them sufficient accuracy for the most important practical purposes. It is not necessary, in this case, to write a column formula from a theoretical standpoint, but from the standpoint of these tests; otherwise, what value have they?

The straight-line formula was not established from a theoretical standpoint. It was first obtained by plotting the results of tests of full-size columns within the limits of column ratios of about 40 and 120, then drawing a straight line which would best represent those results, in the process of determining the corresponding numerical quantities required. It is true that some attempts have been made to establish the straight-line formula analytically, but it is an empirical formula, pure and simple; and all such attempts are defective and without analytic merit.

If a straight line be drawn through the centers of the three groups of the results of tests shown on Fig. 10 for the column ratios of 50, 85, and 120, as shown by the broken line, ignoring for the moment the three results belonging to the Type *B* cross-section, that line may

Mr. Burr.

be represented by the following equation, p representing the average ultimate resistance, in pounds per square inch:

$$p = 36\,550 - 71\,\frac{l}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This equation holds true for all values of the column ratio between 50 and 120; indeed, it may be taken as essentially correct between 40 and 130.

This formula represents fairly the mean ultimate resistances for the different columns tested. There is a not-well-defined zone of rapid distortion of each column immediately below the ultimate resistance, which cannot be considered as of much resisting value, although the load is constantly increasing, up to the point of failure. By examining the diagrams published in the report, this zone may be taken (for illustration) at about 15%, as an average, of the ultimate resistances. Then, if 85% of Equation (1) for p is taken, and if one-half of that value is considered a safe working stress, which is reasonable, the formula for that working stress will become

$$p = 15\,500 - 30\,\frac{l}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If the column ratio is 50, the working stress becomes 14 000 lb. per sq. in.; or, if the column ratios are 85 and 120, respectively, the working stresses become 12 950 and 11 900 lb. per sq. in., respectively. These values are certainly reasonable and safe; in fact, it may be said that they are rather below than above the limit of safe values. They are not given here, however, as values that should be adopted, but as illustrating the proper method of procedure to determine safe working stresses. They show that 12 000 lb. per sq. in. is not justified by these tests; and they show, further, that it is irrational and also unjustifiable to take the same unit working resistance for column ratios of 50 and 85. Finally, they certainly show, also, that it is far from useless for the Committee to attempt to write a rational column formula, either from a theoretical, or from some other, standpoint, well adapted to the purpose. If, however, it is not thought desirable to write a rational column formula from a "theoretical standpoint", the preceding operation demonstrates conclusively the possibility of writing such a formula entirely from an empirical standpoint, by the use of these test results.

A variation of even 28% in the ultimate tensile resistance of the material has no essential bearing on this point. A proper formula will represent a general run of material, and, as has been pointed out already, it is a simple matter to adjust or modify the formula for extreme cases of any kind. Indeed, there is no better way of doing this than by the aid of a formula based on the tests of columns in which the variations are found.

Mr. von Fabrice.

R. von Fabrice,* Esq. (by letter).—The report of the Special Committee on Steel Columns and Struts is based on results obtained from a series of 213 tests made by the Bureau of Standards at Washington. The tests include only columns with square ends and having sections designed to avoid the necessity of lattices or battens. From the results obtained, the Committee recommends a maximum working unit stress of 12 000 lb. per sq. in. of section on all solid columns with square ends, with $\frac{l}{r} = 80$, or less, and a reduction of the unit stress per square inch, by a straight-line law, from 12 000 lb. per sq. in. for $\frac{l}{r} = 80$, to 8 000 lb. per sq. in. for $\frac{l}{r} = 120$. It must be remembered that this recommendation is only for static loads; special percentage and other allowances in loading must be made for impact and rolling loads.

No account is taken of the following unknown factors which are ever present in all kinds of structures:

Material:

(*a*)—Quality of steel.—High and low carbon.—Amount of work received.

(*b*)—Uniformity.—Segregations.—Hard and soft spots.

(*c*)—Workmanship.—Initial bends.—Inadequate or poor riveting.

Condition of Loading:

(*a*)—End bearing.—Pin or square ends.—Ends not milled perpendicular to axis.

(*b*)—Eccentricity.—Load not applied at axis of column.

(*c*)—Transverse loading.—Combined beam and column action.

(*d*)—Repeated stresses.—Loads repeated and perhaps alternated many times.

Use:

(*a*)—Overloading.—Impossibility of always determining exact loading.

(*b*)—Secondary stresses.—Sometimes a considerable proportion of the primary stress.

(*c*)—Deterioration or corrosion.—Exposure to corrosive elements.

(*d*)—Facilities for inspection.—Forms of columns which may be frequently and easily examined.

Fig. 11 shows the relation between various column formulas and that recommended in the report. It also shows the great reduction in unit loading as recommended by the Committee for values of

* Chf. Draftsman, Public Service Elec. Co., Newark, N. J.

$\frac{l}{r} = 80$, or less, and its relation to other formulas previously used for column design. Mr. von Fabrice.

The maximum value of 12 000 lb. per sq. in. of cross-section, based on tests made only for solid sections of perfect fabrication, workmanship, and extremely small variations in chemical and physical requirements, seems ultra-conservative, especially as practically all previously erected structures have been designed with values as high as 14 000 to 16 000 lb. per sq. in. as a maximum, as column failures are practically non-existent, and as there are no records available of failures of columns which were properly designed according to previously accepted formulas.

Practical experience on existing structures shows defective workmanship, such as defects in bearing, riveting, and alignment, and it is very doubtful whether a column has ever been put in place with every rivet tight and filling out the hole, without kinks or twists, with perfect alignment, uniform in material, and with 100% bearing contact at the splices.

The tests conducted by the Committee do not include latticed sections. Eccentric loading and transverse loading, which tend to reduce the strength of the structure, or increase its cost in proportion to the decrease in allowable stresses in the materials, have not been investigated.

The materials and workmanship as specified for the columns tested were far above normal. The inspection as to materials and workmanship was also more rigid, and the most careful selection of quality of specimen, that is, uniformity of metal, was obtained, which is far from practical conditions. The sections tested were over-riveted throughout, that is, a maximum riveting spacing of 4 diameters of rivet was used. The bearing surfaces were scraped so as to be as nearly perfect as possible, insuring uniform bearing throughout, a condition never obtained in practice. All computations for area of section were made by calibration, and not, as in practice, for nominal area, which amounted to as much as 4% from the nominal area.

The Committee established a working limit for the steel, which it termed the Useful Limit Point. This Useful Limit Point is taken to represent the point where the metal ceases to have positive structural value, and is somewhat below the ultimate unit strength of the material and slightly above the region of perfect elasticity.

The results of the tests made for Columns 5 and 5*B* are very interesting, as they show that the commonly held opinions that the failure of columns is due to the crippling of the outstanding legs, and that an angle $\frac{1}{2}$ in. thick would be more than twice as strong as one $\frac{1}{4}$ in. thick, are erroneous.

Mr. von Fabrice.

The tests on Column 5 were for 8-in. H-columns of 32 lb. per ft., having an area of 9.17 sq. in., and that for Column 5*B* was on 8-in. H-columns of 91 lb. per ft., with an area of 26.64 sq. in. The average Useful Limit Point for the first three tests of Column 5, light section, was found to be 34 700 lb. per sq. in.; for the second, Column 5*B*, it was, for the first three tests, 19 700 lb. per sq. in., or a variation of 15 000 lb. per sq. in. between the light and heavy sections, or, in other words, the heavy sections were found to be only 57% as good, per unit of area, as the lighter sections.

This variation in unit strength is accounted for by the Committee as follows:

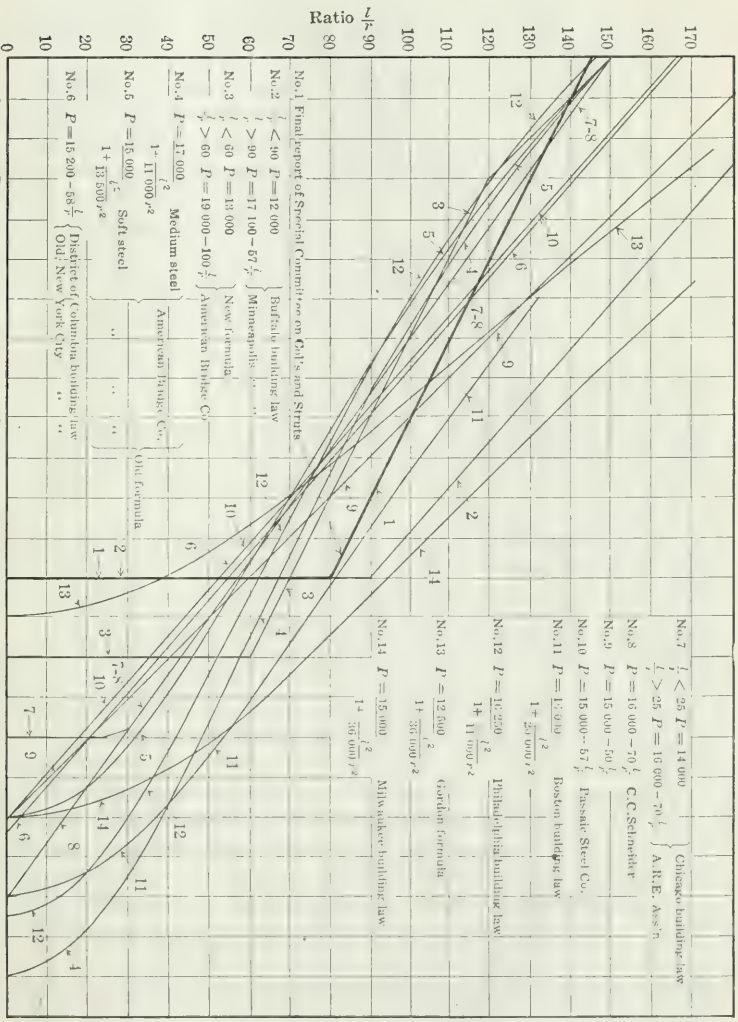
"The physical characteristics and strength of structural steel are affected by the amount of working which it receives in passing through the rolls, and, for the same size of ingot, the heavy material does not receive as much working as the light; consequently, for the same chemical composition, the heavier section is weaker."

This, however, was thought by the Committee to have been overcome by changing the carbon content at the mill.

If this is true, then the variation of the Useful Limit Point for Columns 5 and 5*B* shows that great variations in the strength of materials are prevalent, as all the materials used were selected from supposedly the best that the mills could produce as to uniformity of product. Such being the case, it is evident that the present process of manufacture of steel is not what it should be, and that the mills do not produce uniform metal. This would be complicated further by the fact that the present accepted commercial methods of testing do not show such variations, and, therefore, do not give correct indications as to the behavior of the steel under compressive loads. The tests at the mill indicated no such possible variation, because all the materials accepted by the Committee were practically within the limits of the specifications adopted by it, which were identical with the ordinary 60 000-lb. bridge steel, excepting that greater uniformity was required in both chemical and physical qualities.

The difference in strength of Columns 5 and 5*B* is more probably due to the fact that the distribution of the metal in the lighter H-section is better than in the heavy H-section, in relation to the neutral zones of the sections. This is contrary to the explanation given by the Committee. In built-up sections, the increase in metal is farther from the neutral zones than in H-sections, for the increase in weight and, consequently, the distribution of metal is in favor of the lighter column sections. If this were not the case, the box column and other sections tested should show results similar to those for Columns 5 and 5*B* for their respective light and heavy sections. Therefore, the explanation of the Committee for the behavior of Columns 5 and 5*B*, as previously mentioned, is not correct, because the strength and

Mr. von Fabrice.


Ratio $\frac{l}{k}$
No.1 Final report of Special Committee on Coal's and Struts
No.2 $\frac{l}{r} < 90$ $P = 12\,000$
$\frac{l}{r} > 90$ $P = 17\,100 - 67\frac{l}{r}$ Buffalo building law
No.3 $\frac{l}{r} < 60$ $P = 13\,000$
$\frac{l}{r} > 60$ $P = 19\,000 - 100\frac{l}{r}$ Minneapolis " " New formula
No.4 $P = \frac{17\,000}{1 + \frac{l^2}{11\,000\,r^2}}$ Medium steel American Bridge Co. American Bridge Co. Old formula
No.5 $P = \frac{15\,000}{1 + \frac{l^2}{13\,500\,r^2}}$ Soft steel " " " "
No.6 $P = 15\,200 - 68\frac{l}{r}$ District of Columbia building law Old N. York City " "
No.7 $\frac{l}{r} < 25$ $P = 14\,000$
$\frac{l}{r} > 25$ $P = 16\,000 - 70\frac{l}{r}$ Chicago building law A.R.E. Assn
No.8 $P = 16\,000 - 70\frac{l}{r}$ C.C. Schneider
No.9 $P = 15\,000 - 50\frac{l}{r}$ Passaic Steel Co.
No.10 $P = 15\,000 - 57\frac{l}{r}$ Passaic Steel Co.
No.11 $P = \frac{15\,000}{1 + \frac{l^2}{25\,000\,r^2}}$ Boston building law
No.12 $P = \frac{15\,000}{1 + \frac{l^2}{11\,000\,r^2}}$ Philadelphia building law
No.13 $P = \frac{12\,000}{1 + \frac{l^2}{36\,000\,r^2}}$ Gordon formula
No.14 $P = 15\,000$ $1 + \frac{l^2}{36\,000\,r^2}$ Milwaukee building law

Thousands of Founds per Square Inch.

FIG. 11.

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uniformity of the steel for the other heavy built-up sections are likewise influenced by the amount of working that the metal receives in passing through the rolls and, therefore, should show results similar to those for Columns 5 and 5*B*.

The report of the Special Committee does not recommend economy either in labor or material, nor does it suggest any means of improving the design of the columns.

The recommendations of the Committee, of using 12 000 lb. per sq. in. as a maximum, if accepted by the Profession as standard for column design, will tend to increase the costs of all construction; and this does not seem to be justified, in view of the great quantity of existing work based on higher stresses and giving satisfactory results.

To use the lower stresses recommended by the Committee might be justifiable for long columns which are subjected to the full 100% of the maximum calculated column loading. This condition of column loading, however, is the exception rather than the rule, and, as it is doubtful whether a column ever will receive the maximum load from all sources at any one time, a large factor of safety (partly unknown) is introduced automatically into the design.

The Useful Limit Point for square-ended, solid-section columns was specified by the Committee at 24 000 lb. per sq. in., therefore, the use of 12 000 lb. per sq. in. as a maximum will give an actual safety factor of two.

Previous column formulas, based on an ultimate compressive strength of from 60 000 to 64 000 lb. per sq. in., and allowing from 15 000 to 16 000 lb. per sq. in. as a maximum, did not permit a factor of safety of four within the "positive structural" value of the steel.

Using the value, 12 000 lb. per sq. in., as recommended by the Committee, and conceding the fact of its correctness, the former maximum stress of 16 000 lb. was really 75% of the reliable strength of the structure, allowing in reality only 8 000 lb., or 50% for safety, instead of, as formerly assumed, 300% of additional strength. Considering the actual practical variations in workmanship, bearing, riveting, etc., it is fair to assume that, for columns designed for a maximum stress of 16 000 lb. per sq. in., the remaining allowable actual 50% is easily absorbed; therefore, the former stresses used in column design do not give any theoretically available actual factor of safety, and the actual factor of safety in existing structures is obtained solely by the condition of partial loading, as mentioned previously.

The limiting value of 12 000 lb. per sq. in. for maximum stress, for columns with ratios of $\frac{l}{r} = 80$, or less, is not verified or justified by observations on existing structures which were designed for higher values; therefore, the Committee's recommendation does not confirm the experiences of actual practice.

From the report the following deductions can be made: Mr. von Fabrice.

1. That the present commercial, physical, and chemical tests are unreliable for correct indications as to the behavior of metal under practical loading, where used as columns or struts.

2. That the present process of manufacture of steel does not produce metal which is of uniform strength, even though the present commercial tests would indicate it to be of such strength.

3. That heavy sections are weaker per unit of area than those of thinner metal.

4. That the actual excess strength in existing structures is nearer 50% than 300%, as formerly assumed, or a factor of safety of one-half instead of four.

5. That the statement: "the form of section has no material influence on the strength of the column", is disproved by tests on Column 5 and 5*B* only.

6. That bulb angles in columns are no stronger than plain angles of equal section.

7. That the unit stresses as formerly used are excessive, and in no case should a greater loading than 12 000 lb. per sq. in. be used.

8. That the statement: "Ratio of thickness to width of outstanding flanges is not the controlling feature for strength of a column", is disproved by Columns 5 and 5*B* only.

9. That the physical characteristics and strength of structural steel are uncertain quantities (if the Committee's explanation for behavior of Columns 5 and 5*B* is accepted). Other tests show only slight variation.

10. That the recommendations were based on the tests made under the most favorable conditions as to workmanship and material, conditions which do not obtain in practice.

11. That the former safety factor of four, as applied to the ultimate tensile strength, is equivalent to a factor of about two, based on the yield point.

12. That, if the columns are to be as strong as tension members, they must be designed with a factor of safety of two, on a basis of Useful Limit Point.

13. That some basis of specifying the amount of working that the metal should receive per unit of volume, in its transformation from ingot to finished shape, should be devised.

14. That the Useful Limit Point of steel, as established by the Committee, gives a more tangible or serviceable indication of the ultimate behavior of the metal, and bears a definite and fairly constant relation between the results of the final and specimen tests.

L. J. Mensch,* M. Am. Soc. C. E. (by letter).—The Committee has presented to the Society the most elaborate series of tests on Mr. Mensch.

* Chicago, Ill.

Mr. Mensch.

flat-ended steel columns which were ever made. It is quite a disappointment that these modern tests, made under extremely favorable conditions, often did not show up better than, and sometimes not as well as, the tests on pin-ended iron columns, made by the Watertown Arsenal more than 30 years ago. Under these circumstances, one could not expect the Committee to recommend an increase in the customary working stresses—desirable as this would have been in these times of increasing cost of all construction work.

The Committee has pointed out that, with respect to the transmission of the loads at its ends, the practical column is loaded under conditions differing widely from those of scraped ends in the testing machine.

The writer is really surprised that, during the progress of the tests, no attempt was made to test one column out of three with ends as they came from the bridge works; if this had been done for the slenderness ratios of 120 and 155, the Committee would have found the proper justification for its ruling to allow a working stress of 8 000 lb. per sq. in. for $\frac{l}{r} = 120$, which is a drop of 33% below the working stress of a column of $\frac{l}{r} = 80$; this, at first glance, would lead one to believe that the ruling is not in accordance with the tests, which showed a drop in strength of only 10% for columns having a slenderness ratio of $\frac{l}{r} = 120$ below the ultimate strengths of columns of $\frac{l}{r} = 85$.

Long columns with imperfect flat ends behave mostly like columns with frictionless pins, but short columns behave like eccentrically-loaded columns. This fact was impressed on the writer by his experience with wooden struts in reinforced concrete form work, and was clearly proved to him during a visit to the Pittsburgh Laboratory of the Bureau of Standards. Mr. P. H. Bates, in charge of the Laboratory, had the kindness to make the following test, in the presence of W. K. Hatt, M. Am. Soc. C. E., and the writer: A sound piece of 2 by 4-in. spruce scantling, 12 ft. long, stored under shelter for many months, was cut off by hand to a length of 6 ft. and placed in the 600 000-lb. machine, between wedges at the bottom and a piece of timber at the top. The maximum load was 4 000 lb., and the deflection about 2 in. The actual dimensions of the strut were only $1\frac{3}{4}$ by $3\frac{5}{8}$ in., hence the ultimate strength was 630 lb. per sq. in. According to the Watertown Arsenal reports for 1884, square-ended struts of green, white pine, of the same slenderness ratio as the spruce strut, failed at 1 700 lb. per sq. in. The two values of 630 and 1 700 show the difference in strength of the practical wooden column and the

column with perfectly square ends in a testing machine. Here is the explanation as to why the standard textbooks advise the use of a factor of safety of from 6 to 10 for wooden columns. Such high factors of safety have properly been nicknamed "factors of ignorance." Mr Mensch.

Iron or steel columns with ideal pin ends and ideal flat ends do not show such a large difference in strength for the slenderness ratios in practical use, but the Committee did not instruct us in this respect. It realized the demand of engineers for a rational column theory, yet, on account of the non-uniformity of commercial grades of steel, and the great influence of the amount of rolling the various sections receive and the resulting difference in the strength of the columns, deemed it advisable not to attempt the task.

Although there was a great difference in strength in light, medium, and heavy-weight sections, the drop in strength in each type, from the comparatively short column having slenderness ratios of $\frac{l}{r} = 50$ to $\frac{l}{r} = 155$, was remarkably uniform, and it is this decrease of strength which engineers want to have explained, it being known for many years that the grade of steel and the amount of rolling greatly affect its strength, and it being quite common practice, in reinforced concrete, for example, to allow a higher stress on wires than on bars from which the wires are made.

This deficiency in the report of the Committee the writer will endeavor to supply. A solution of the problem cannot be found by adhering to the common theory of flexure as propounded in the textbooks in the stereotyped form:

1 The modulus of elasticity is constant;
2 A plane section before bending remains plane after bending;
3 The direct stresses vary uniformly across the section in all cases (Hooke's law);
4 Calculations are made with references to working stresses and safe loads, rather than with references to ultimate stresses and ultimate loads.

One does not know what "proper working stresses" are, unless one makes tests to destruction or undue distortion; therefore, Point 4 leads only to much speculation and many exercises in mathematics, without being conducive to an exact understanding of the strength of materials.

Point 1 has been disproved by all careful observers, and by Figs. 5 and 6 and Plate XXXIII of the Committee's report.

Point 2 is disproved by Plates XXXI and XXXII, and was never proved by anybody for high stresses.

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Hooke's law is acknowledged to be only an approximation for steel at low stresses, is certainly far from being correct for high stresses, and is never true for concrete, wood, or cast iron, for any stage of loading.

A correct solution of the beam or column theory can only be found by examining the conditions at the time of failure or just before it.

For this condition the writer makes the following assumption: A column fails when, in any cross-section, the extreme fibers on one side of a cross-section are stressed beyond the yield point and are subjected to a strain which is from 50 to 100% greater than the deformation just below the yield point. Following the lead of the Committee, this maximum strain will be called the Useful Limit Strain (U. L. S.), and the corresponding stress the Useful Limit Point (U. L. P.), which stress is not identical, however, with the point adopted by the Committee.

The Committee did not furnish stress-strain diagrams for the separate angles, plates, or beams, but only for the built-up columns up to failure, which diagrams are somewhat clouded by the influence of the rivet holes and the bending of the columns. Figs. 5 and 6 and Plate XXXIII show such diagrams, and they can be compared in shape to cubic (or higher) parabolas. Even with the not strictly correct assumption of a plane cross-section remaining plane after deformation, one can readily see that, in any cross-section of a bent column, where the extreme fibers are strained to the U. L. S., the fibers which are quite distant from them, although strained quite considerably less, are subjected to stresses only slightly less than the U. L. P., and very much more than the old straight-line theory would warrant us to assume.

The second assumption, then, is, that a column fails when, in any cross-section, the extreme fibers on one side are stressed to the U. L. P.; and the distribution of the unit stresses over the section follows the law of a cubic parabola. The cubic parabola was adopted because it gave a good and sufficient agreement with tests, and is simpler in application than any higher curve. Fig. 12 shows the outline of such a parabola.

Let f_c represent the rise of the parabola and the U. L. P.;
" kd represent the semi-chord and the distance of the neutral axis from the extreme fiber in compression;

then the equation of the parabola is $y^3 = \frac{(kd)^3}{f_c} x$

or $x = \frac{y^3}{k^3 d^3} f_c$ ..... (1)

or the stress, f^y, at any point, y, equals $f_c - x = f_c \left(1 - \frac{y^3}{k^3 d^3}\right)$ .. (2)

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The area, OAB, represents the sum of all stresses between the neutral axis and the extreme fibers, and equals $\frac{3}{4}\ b\ kd\ f_c$.....(3)

when the width of the section is constant and equals b.

The area, $OCA = \frac{1}{4}\ b\ kd\ f_c$.....(4)

The distance of the centroid of OAB from $OB = 0.4\ kd$.(5)

" " " " " " OCA " " $= 0.8\ kd$.(6)

The tangent at A intersects the x axis at $2\ f_c$ from O.

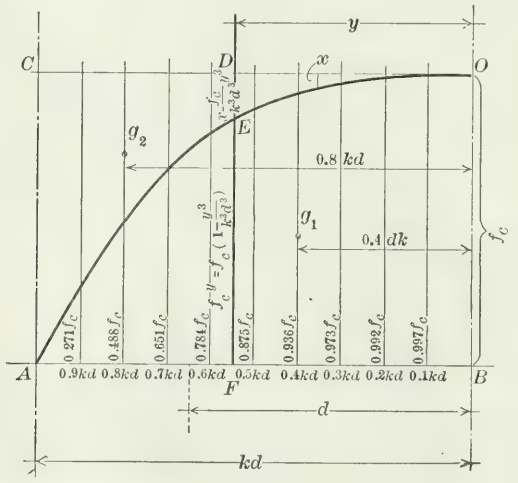

FIG. 12.

The writer developed the theory of long columns by first solving the problems of the short eccentrically-loaded column. After he found a substantial agreement between his theory of eccentrically-loaded columns with actual tests to destruction, he found a simple method to determine the sidewise deflection under which long columns fail, and considered a long column at the moment of failure as a short column with a known eccentricity.

Fig. 13 illustrates short, eccentrically-loaded columns. Where the entire cross-section of the column is in compression, as shown in Fig. 13 (*a*), the neutral axis is outside of the cross-section, and the value of k is greater than 1.

By increasing the eccentricity we may reach a case where the neutral axis coincides with one edge of the section, such as shown in Fig. 13 (*b*). Here k equals 1.

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For larger eccentricities, the neutral axis will fall inside of the section, as shown in Fig. 13 (*c*), and k becomes smaller than 1.

For a rectangular cross-section, it is a very simple matter to find a relation between the ultimate load, its eccentricity, the dimensions of the cross-section, and the U. L. P.

For Fig. 13 (*a*) we easily find that the sum of all stresses,

$$W = b \times \text{area } OBDF - b \times \text{area } OED,$$

$$\text{or} \quad W = b \times d\, f_c - b \times \frac{1}{4} d \times f_c \times \frac{1}{k^3} = b\, d\, f_c \left(1 - \frac{1}{4\, k^3}\right). \quad (7)$$

The moment of the load, W, about the extreme fibers in compression,

$$M = b \times \text{area } OBDF \times \frac{1}{2} d - b \times \text{area } OED \times 0.8\, d$$

$$= b\, d^2\, f_c \left(\frac{1}{2} - \frac{0.2}{k^3}\right) \cdots (8)$$

The distance of W from the extreme fibers in compression, $m = \frac{M}{W}$, and the eccentricity under which the column fails,

$$e = \frac{1}{2} d - m = \frac{0.075\, d}{k^3 - \frac{1}{4}} \cdots (9)$$

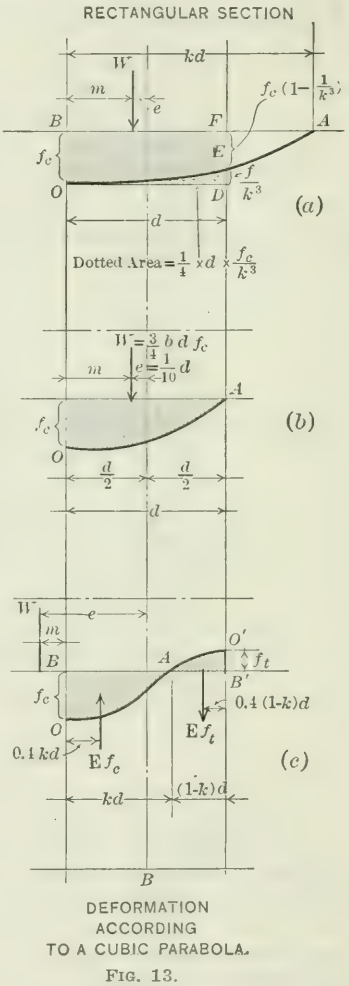


FIG. 13.

For $k = 1$, Equations (7) to (9) simplify to

$$W = \frac{3}{4}\, b\, d\, f_c, \quad M = 0.3\, b\, d^2\, f_c, \text{ and } e = 0.1\, d. \ldots\ldots (10)$$

For kd smaller than d, or k smaller than 1, we have to distinguish two cases. In one case, as in brick or other masonry columns and in foundations, no tensile stresses can be counted on, and the distribution of stresses will be according to Fig. 14. W equals, again, the area,

Mr. Mensch.

OAB, multiplied by b, and the location of W is given by the centroid of the parabola.

$$W = \frac{3}{4}\, b\, kd\, f_c, \text{ and}$$

$$e = \frac{1}{2}\, d - m = \frac{1}{2}\, d - 0.4\, kd \ldots (11)$$

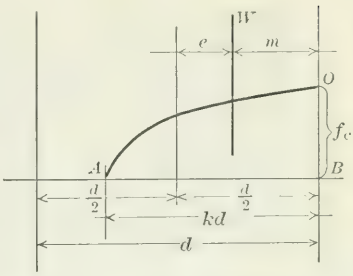


FIG. 14.

It is quite evident that, in this case, W can at no time fall outside of the section. Materials which possess tensile strength will allow W to fall outside, in which case the effect of the eccentric loading will approach girder action. For this case we can only make our theory agree with tests by making a third assumption, namely, that the extreme fibers in tension and compression attain the U. L. P. simultaneously at the time of failure.

From Fig. 13 (c) we have $W + \Sigma f_t = \Sigma f_c$,

or $$W = b \times \text{area } OAB - b \times \text{area } O'AB',$$

$$W = b\,\frac{3}{4}\, kd\, f_c - b\,\frac{3}{4}\,(1-k)\, d\, f_t = \frac{3}{4}\, b\, d\, f_c \left\{ k - (1-k)\,\frac{f_t}{f_c} \right\} \ldots (12)$$

$$M = b \times \text{area } OAB \times 0.4\, kd - b \times \text{area } O'AB' \times \left\{ d - (1-k)\, 0.4\, d \right\}$$

$$M = \frac{3}{40}\, b\, d^2\, f_c \left\{ 4\, k^2 - \frac{f_t}{f_c}\,(6 - 2\, k - 4\, k^2) \right\} \ldots\ldots (13)$$

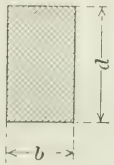
$$m = \frac{M}{W}, \text{ and } e = \frac{1}{2}\, d - m.$$

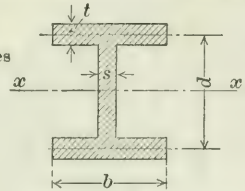
Where M becomes negative, m is negative, and it means that the point of application of W falls outside of the section. The ratio of $\frac{f_t}{f_c}$ is about 1 for structural steel, $\frac{1}{10}$ for concrete, $\frac{1}{5}$ for cast iron, and 1.25 for green timber.

Equations (7) to (13) enable us to find W and e for rectangular sections when k is known. By giving k various values from 3 to 0.6, for example, we may obtain a series of values of W and e which will include nearly all cases found in practice. This has been done, and the results may be found in Table 25, Columns 1 and 2. For example, for $k = 1.5$, it is found that $W = 0.926\, b\, d\, f_c$, and $e = 0.024\, d$. The short concentrically-loaded column can carry the load, $b\, d\, f_c$; therefore, the eccentrically-loaded column carries in this case only 92.6% of that load.

The tests by the Committee were made on sections which in most cases can be considered as being composed of rectangles. For the purpose of a simplified theory, we will idealize the sections, like those shown by Figs. 15 and 16. Let A, the entire cross-sectional area, $= 2\, b\, t + d\, s$.

TABLE 25.—Ultimate Loads and Eccentricities, at Failure, of Structural Steel Columns.





Column is assumed to bend at right angles to x-axis

Area of Flange $= b\,t$
" " Web $= d\,s$
Full Area $= 2\,b\,t + d\,s = A$
$\frac{bt}{A} = n$, $\frac{ds}{A} = n_1$

				$n = 0.25$ $n_1 = 0.50$			$n = 0.290$ $n_1 = 0.42$			$n = 0.333$ $n_1 = 0.333$			$n = 0.375$ $n_1 = 0.25$			$n = 0.400$ $n_1 = 0.20$		
	Radius of gyration			$0.407\,d$			$0.425\,d$			$0.443\,d$			$0.456\,d$			$0.465\,d$		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
K	$\frac{W}{b\,d\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$	$\frac{W}{A\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$	$\frac{W}{A\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$	$\frac{W}{A\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$	$\frac{W}{A\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$	$\frac{W}{A\,f_c}$	$\frac{e}{d}$	$\frac{e\,k}{d}$
3.0	0.991	0.003	0.009	0.986	0.006	0.018	0.985	0.006	0.018	0.985	0.007	0.021	0.984	0.008	0.024	0.984	0.009	0.025
2.0	0.969	0.010	0.020	0.953	0.022	0.013	0.951	0.023	0.046	0.948	0.024	0.047	0.945	0.028	0.056	0.944	0.029	0.057
1.50	0.926	0.024	0.036	0.889	0.055	0.082	0.883	0.058	0.087	0.877	0.065	0.098	0.871	0.070	0.105	0.867	0.074	0.111
1.30	0.886	0.038	0.049	0.829	0.090	0.116	0.820	0.097	0.126	0.810	0.108	0.140	0.801	0.117	0.152	0.795	0.122	0.159
1.10	0.812	0.069	0.076	0.718	0.170	0.187	0.703	0.187	0.206	0.687	0.209	0.230	0.671	0.230	0.253	0.662	0.244	0.269
1.00	0.750	0.100	0.100	0.625	0.260	0.260	0.605	0.228	0.288	0.583	0.329	0.329	0.563	0.370	0.370	0.550	0.391	0.391
0.85	0.638	0.160	0.136	0.263	1.24	1.05	0.221	1.60	1.36	0.175	2.20	1.87	0.131	3.15	2.68	0.105	4.10	3.49
0.80	0.600	0.180	0.144	0.225	1.50	1.20	0.189	1.91	1.53	0.150	2.60	2.08	0.113	3.71	2.98	0.090	4.76	3.81
0.70	0.525	0.220	0.154	0.150	2.33	1.64	0.126	2.98	2.09	0.100	4.00	2.80	0.075	5.65	3.96	0.060	7.35	5.15
0.60	0.450	0.260	0.156	0.075	4.79	2.87	0.063	6.05	3.10	0.050	8.10	4.86	0.038	11.5	6.90	0.030	14.80	8.88

W = Ultimate load ; A = Area of cross-section ; f = Useful Limit Point ; e = Eccentricity.

Mr. Mensch.

Let $\frac{b\,t}{A} = n$, $\frac{d\,s}{A} = n_1$, and $f_t = f_c$. Mr. Mensch.

For bending under right angle to the X-axis and for $k > 1$, we easily find, considering Equations (7) and (8):

$W = b\ t\ f_c + d\ s\ f_c \left(1 - \frac{1}{4\,k^3}\right) + b\ t\ f_c \left(1 - \frac{1}{k^3}\right)$, which easily reduces to

$$W = A\ f_c \left(1 - \frac{n + \frac{n_1}{4}}{k^3}\right) \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

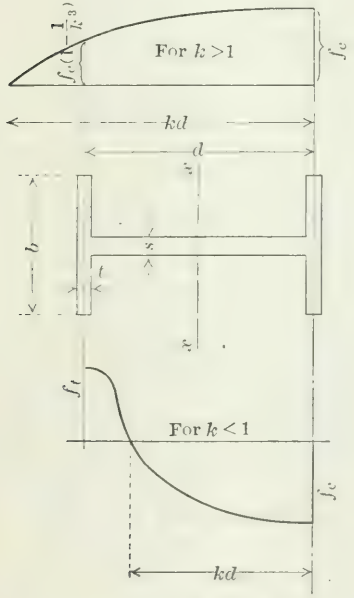


Fig. 15.

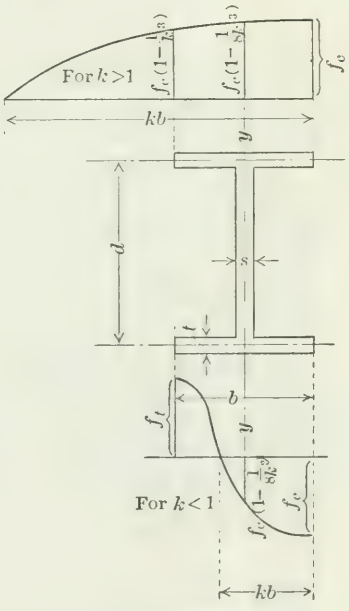


Fig. 16.

$$M = s\,d^2\,f_c \left(\frac{1}{2} - \frac{0.2}{k^3}\right) + b\ t\ f_c \left(1 - \frac{1}{k^3}\right) d,$$

$$M = A\ d\ f_c \left(\frac{1}{2} - \frac{n + 0.2\,n_1}{k^3}\right) \ldots\ldots\ldots\ldots (15)$$

For $k < 1$, $W = b\ t\ f_c + s\ k\,d \frac{3}{4} f_c - b\ t\ f_t - s\,(1-k)\,d \frac{3}{4} f_t$,

for $f_t = f_c$, this reduces to $W = \frac{3}{4}\ s\ d\ f_c\ (2\,k - 1) \ldots\ldots (16)$

$M = \frac{3}{4}\ s\ kd\ f_c \times \frac{4}{10}\ d - b\ t\ f_t\ d - \frac{3}{4}\ s\ d\ f_t\ (1-k) \times \frac{1}{2} d - 0.4\,(1-k)\,d\,\frac{1}{2}$

for $f_t = f_c$, $M = 0.6\ s\ d^2\ f_c \left(k^2 + \frac{1}{4}\ k - \frac{3}{4} - 10\ \frac{n}{6\,n_1}\right) \ldots\ldots (17)$

Mr. Mensch.

For bending under right angle to the Y-axis, as shown in Fig. 16, and for $k > 1$:

$$W = 2\,b\,t\,f_c\left(1 - \frac{1}{4\,k^3}\right) + s\,d\,f_c\left(1 - \frac{1}{8\,k^3}\right)$$

$$= A\,f_c\left(1 - \frac{4\,n + n_1}{8\,k^3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$M = 2\,t\,b^2\,f_c\left(\frac{1}{2} - \frac{0.2}{k^3}\right) + s\,d\,f_c\left(1 - \frac{1}{8\,k^3}\right) \times \frac{1}{2}\,b.$$

$$M = A\,b\,f_c\left(\frac{1}{2} - \frac{6.4\,n + n_1}{16\,k^3}\right) \quad \ldots\ldots\ldots\ldots (19)$$

For $k < 1$, and considering Equations (12), (13), and (2):

$$W = 2 \times \frac{3}{4}\,b\,t\,f_c\,(2\,k - 1) + d\,s\,f_c\left(1 - \frac{1}{8\,k^3}\right)$$

$$= A\,f_c\left(3\,k \times n - 1 \times 5\,n + n_1 - \frac{n_1}{8\,k^3}\right) \quad \ldots\ldots\ldots (20)$$

$$M = \frac{3}{40} \times t\,b^2\,f_c\,(8\,k^2 + 2\,k - 6) \times 2 + d\,s\,f_c\left(1 - \frac{1}{8\,k^3}\right)\frac{1}{2}\,b.$$

$$M = A\,b\,f_c\left(1.2\,k^2 \times n + 0.30\,k\,n - 0.9\,n + \frac{1}{2}\,n_1 - \frac{n_1}{16\,k^3}\right) \ldots (21)$$

Equations (16) and (17), also (20) and (21), cannot be used for $k = 1$, because the assumption was made that the full U. L. P is acting on the tension side. It is very probable that the neutral axis must be at a fixed proportion of the depth distant from the extreme fibers in tension before these fibers are stressed to the U. L. P. The writer believes that this is the case for values of k smaller than 0.9 or 0.85. When W and M are known, we easily find $m = \frac{M}{W}$ and $e = \frac{1}{2}\,d - m$.

These values have been computed for various values of k and n, and may be found in Table 25, Columns 4 to 17. The values of n were selected in conformity with the specimen tested by the Committee.

For example, Type 1 corresponds to $n = 0.272$
" 1A " " $= 0.282$
" 2 and 2A " " $= 0.33$
" 3 " 3A " " $= 0.40$
" 5 " " $= 0.38$
" 5A " " $= 0.40$

To the right of the columns containing the figures for $\frac{e}{d}$ in Table 25 is found another series of figures which are obtained by multiplying

k by $\frac{e}{d}$, giving the values of $\frac{e\,k}{d}$, which are of great importance in the theory of long columns, as will be shown presently. Mr. Mensch.

Theory of Long Columns.—It is known that long columns which are concentrically loaded at knife-edge points fail by buckling. This buckling is the result of some interior eccentricity which, as a rule, is not sufficiently large to produce a noticeable deflection at low loads; but, when the loads become high, especially after the yield point is exceeded in the extreme fibers of a section, the moments are sufficiently large to produce an appreciable deflection. This causes the line of application of the load to fall outside of the center line of the column, as shown in Fig. 17, much more than is due to the original eccentricity, and, under this enlarged eccentricity, the column finally fails. The problem of the strength of a long column is thus reduced to finding the eccentricity of the deflected column. This, of course, is very difficult, because we do not know how large the interior eccentricity is nor how it is distributed, and because we are not yet in possession of the laws governing the flow of the materials after the yield point is exceeded, or of the laws of the deformation of a cross-sectional plane. In the practical column the deflection is so small, in comparison with the length of the column, that we cannot go far out of the way in assuming that the curve of the elastic line is a parabola or a part of a circle. If the radius of the circle is known we can find at once the rise of the curve, and, therefore, the eccentricity of the deflected column.

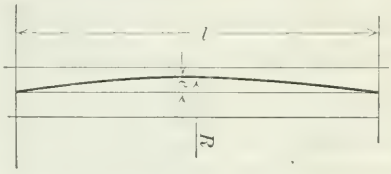


FIG. 17.

Fig. 18 shows a coupon of a column under ultimate load. The original prism, of the length $\Delta\, l$, becomes a bent frustum, and the greatest compressive strain in the extreme fiber equals the U. L. S. (here called ε) multiplied by $\Delta\, l$. We can now find the radius of curvature of the bent column, at the point where the U. L. P. is first reached, by the same reasoning as used in the theory of beams, and can establish the proportion,

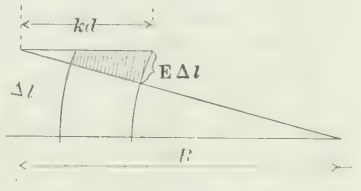


FIG. 18.

$$\frac{R}{\Delta\, l} = \frac{k\,d}{\varepsilon\,\Delta\, l}, \text{ or } R = \frac{k\,d}{\varepsilon} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

but it is known that, in a circle or a parabola with the radius of curvature, R, at the crown, the rise, $e = \frac{l^2}{8\,R}$, or from Equation (22),

Mr. Mensch. $e = \frac{\varepsilon}{8} \frac{l^2}{k\,d}$, which can also be written in the form,

$$\frac{e\,k}{d} = \frac{\varepsilon}{8} \frac{l^2}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The right side of Equation (23) depends only on the known dimensions of the column and on the supposedly known U. L. S., hence $\frac{e\,k}{d}$ can be computed for each column to be investigated. Table 25, however, contains values of $\frac{e\,k}{d}$ for all practical cases; therefore, we can find the strength of a long column in terms of the strength of the correspondingly short column which fails by being stressed to the U. L. P. over the entire section, by computing the value of $\frac{\varepsilon}{8} \frac{l^2}{d^2}$ and by finding in Table 25 the corresponding value for W. The writer has found a good agreement with tests by assuming:

$\varepsilon = 0.0015$ for mild steel, plain concrete, hard brick laid in cement mortar, and for terra cotta block columns;
$\varepsilon = 0.01$ for hooped concrete and for cast iron;
$\varepsilon = 0.004$ for green timber;
$\varepsilon = 0.0026$ for common brick laid in cement mortar.

The conditions of perfectly hinged ends are very rare in a testing machine, as the Watertown Arsenal tests plainly show; such conditions occur, however, in most columns used in building construction, as will be shown later.

The writer will now investigate the behavior of columns with perfect flat ends, such as in the tests which the Committee has presented. As stated before, on account of some slight interior eccentricity, the column will deflect at a certain stage of loading, and, if the ends are not fixed, will try to change their slope, as shown in Fig 19. The platens of the machine being fixed, either by the construction of the machine or by friction, this tendency is overcome by an eccentric application of the load, such as shown in Fig. 20. The form of the elastic line of the bent column with flat ends is similar to that of a uniformly-loaded beam with end restraint. From the ordinary theory of beams, it is known that the moments at the ends are twice as large as in the center. This, however, is only true as long as the yield point is not reached. The fibers of a beam of uniform section will first exceed the yield point at the ends, causing there a

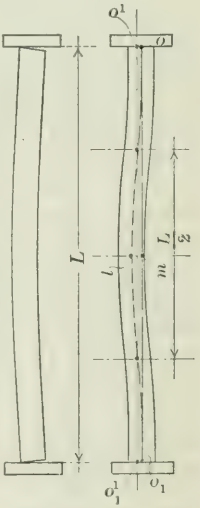


FIG. 19. FIG. 20.

comparatively large deformation, which has the same effect as if the restraint is slightly loosened, thereby allowing the moments in the center to become larger. The few tests which were made on continuous beams, and especially on continuous plates, confirm this reasoning. Probably there is no error, then, in assuming that the new line of application of the load, $o'\ o'_1$, divides the deflection, $l\ m$, in two. It is known, besides, from Euler's theory for very long columns, that this is the case, and that the intersection of the new line of application of the load with the elastic line divides also the column length in two. Mr. Mensch.

Hence, the problem of the strength of a column with perfect flat ends can be reduced to the problem of the strength of a column with perfect hinged ends by assuming the former column to be only of half the length of the latter.

For flat-ended columns, Equation (23) becomes

$$\frac{e\ k}{d} = \frac{\varepsilon}{8}\ \frac{\left(\frac{l}{2}\right)^2}{d^2} = \frac{\varepsilon}{32}\ \frac{l^2}{d^2} \qquad (24)$$

It must be borne in mind that this equation is based on conditions which are not always obtained in the tests. The principal condition is that the moments at the ends shall equal the moment in the center; in fact, the tendency is always there to make the moment at the ends larger. As long as k is greater than 1, every point of the end will be still in contact with the platens of the machine, and the restraint of the ends is bound to be in full action when perfect ends are used; but, where the column is so long that k becomes smaller than 1, only part of the ends will be in contact with the platens, the slope of the ends will change, and we will again approach the condition illustrated in Fig. 19. For the understanding of column action, it must also be remembered that a column with perfectly hinged ends at the time of failure must have a slope at its ends, given by the tangent to the elastic line. It does not make any difference whether we consider the elastic line as a circle or a sine curve, the tangent is given by $\alpha = 3.2\ \frac{e}{l}$, or, substituting the value of e from Equation (23), we obtain

$$\alpha = \frac{3.2}{l} \times \frac{\varepsilon}{8}\ \frac{l^2}{k\ d} = 0.4\ \frac{\varepsilon\ l^2}{k\ d} \qquad (25)$$

Inversely, we can say that a column which has the slope, α, as given by Equation (25), at its ends, must act like a column with ideal hinged ends, regardless of whether it appears to be flat-ended or continuous.

Where the angle at the ends is much smaller than given by Equation (25), the column action lies between that of flat and hinged

Mr. Mensch.

ends. Columns with perfect flat ends are probably extremely rare in actual practice. The rigid heads of the testing machine are infinitely stiffer than the ordinary beam connections in building work. It is known from Euler's theory that ordinary continuation of columns does not make them act as flat-ended. In order to make the column act as a flat-ended one, there must be, at each story, a rigid connection which can transmit the full bending moment, $W e$, of the column to the connecting girders without undue deformation—an extremely rare case in practice.

It is a very difficult matter to prove that the quantities, $\frac{e}{d}$, of Table 25, are in accordance with the tests. By inspecting on Plate XXXII the deflections for the middle of the column, we find that, for a load of 280 140 lb. the deflection is 0.3717 in., and for the ultimate load of 289 800 lb. the deflection is 1.995 in., which means that for an increase of 3% of the load, the deflection increases 150%; therefore, the deflections given in Tables 1 to 5 of the report must be considerably smaller than the deflections found by the writer's theory.

To prove that this theory is sustained by the tests of the Committee, Table 26 has been prepared. Only the tests of columns which have sections in conformity with Figs. 15 and 16 were used. The values of $\frac{e\,k}{d}$ were found by Equation (24), and the U. L. P. of each type was found from the average unit ultimate strength for the slenderness ratio of $\frac{l}{r} = 50$ by dividing the ultimate unit stress by the percentage given in the next column.

The percentage of the strength of the short concentrically-loaded column, which a long column carries, was found from the $\frac{e\,k}{d}$ given in Table 26 by interpolation in the proper columns of Table 25. It is found that this theory agrees with the tests in most cases within 5 per cent.

For very short columns, as, for example, the slenderness ratios of $\frac{l}{r} = 20$, this theory does not seem to apply, on account of the so-called block action, for which, at present, the writer has no explanation to offer.

The Committee discourages the use of longer columns than $\frac{l}{r} = 120$, although its tests on $\frac{l}{r} = 155$ still give very high values. That long columns in actual use are so much weaker than short columns is due to the fact that the girders connecting them exert a moment on such columns, cause them to deflect, and change their slopes

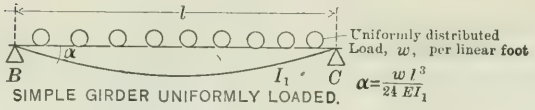
at the ends, and we have, instead of the flat-ended column of the testing machine, a hinged column with eccentric loading. Figs. 21 to 34 explain this point. The indeterminate values of the moments and thrusts were found by the slope and deflection method, and the assumption was made that the slope of a girder at the junction with a column is the same as the slope of the column at the junction. The main point to be learned from these figures is the fact that a moment is exerted by the girders on one or both ends of a column, and that the column deflects either like a hinged column of the length, h, or in some cases of the length, $\frac{3}{4}\,h$, and only in very rare cases of the length, $\frac{1}{2}\,h$. Mr. Mensch.

TABLE 26.—Comparison of Actual and Theoretical Ultimate Loads.

Type.	Value of n.	Length. Feet.	Length. Inches.	Depth of Section, in Inches. d	Depth of Section, in Inches. b	Slenderness Ratio. $\frac{l}{r}$	Slenderness Ratio. $\frac{l}{d}$	$\frac{ek}{d}$, from Equation (24)	Average unit ultimate loads, Bureau of Standards tests.	U. L. P. derived from $\frac{l}{r} = 50$.	Average Unit Ultimate Load from Theory. Percentage of U. L. P.	Average Unit Ultimate Load from Theory. Pounds per square inch.
1....	0.272	9	4	$5\frac{11}{16}$		50	19.7	0.0182	32 700	33 200	0.986	32 700
		15	$10\frac{7}{16}$	$5\frac{11}{16}$		85	33.4	0.052	31 200	33 200	0.946	31 400
		22	$4\frac{13}{16}$	$5\frac{11}{16}$		120	47.2	0.108	28 300	33 200	0.848	28 100
		28	$11\frac{3}{16}$	$5\frac{11}{16}$		155	61.0	0.174	26 200	33 200	0.750	24 900
1 *A*.	0.282	9	10	$5\frac{3}{8}$		50	22.0	0.023	29 200	29 750	0.982	29 200
		16	$8\frac{5}{8}$	$5\frac{3}{8}$		85	37.4	0.066	28 100	29 750	0.920	27 400
		23	$7\frac{1}{16}$	$5\frac{3}{8}$		120	52.7	0.127	25 400	29 750	0.820	24 400
		30	$5\frac{13}{16}$	$5\frac{3}{8}$		155	68.0	0.216	22 700	29 750	0.700	20 900
3....	0.41	9	9	$5\frac{1}{8}$		50	19.0	0.017	34 100	34 600	0.986	34 600
		16	$6\frac{7}{8}$	$5\frac{1}{8}$		85	32.4	0.049	32 400	34 600	0.950	32 900
		23	$4\frac{13}{16}$	$5\frac{1}{8}$		120	45.7	0.098	30 300	34 600	0.884	30 500
3 *A*..	0.40	9	$11\frac{1}{2}$	$5\frac{3}{8}$		50	18.6	0.016	29 500	29 900	0.986	29 500
		15	$11\frac{1}{8}$	$5\frac{3}{8}$		85	31.5	0.047	28 000	29 900	0.958	28 600
		23	$10\frac{3}{16}$	$5\frac{3}{8}$		120	44.5	0.093	26 900	29 900	0.894	26 700
5....	0.38	8	3	.....	8	50	12.3	0.007	38 600	38 800	0.987	38 800
		14	$05\frac{1}{16}$	.....	8	85	21.0	0.021	34 300	38 800	0.951	36 900
		19	$9\frac{5}{8}$	.....	8	120	29.7	0.041	32 000	38 800	0.875	33 800
5 *A*..	0.40	8	$8\frac{1}{2}$	.....	8	50	13.0	0.008	35 400	35 900	0.986	35 400
		14	$9\frac{5}{8}$	.....	8	85	22.2	0.023	32 300	35 900	0.951	34 100
		20	$10\frac{13}{16}$	.....	8	120	31.3	0.046	30 000	35 900	0.872	31 300

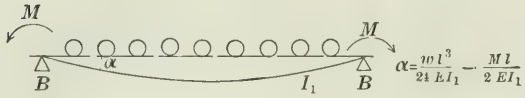
In most cases it will be sufficient to compute a column like a short column, eccentrically loaded by the moment, M, shown in Figs. 21 to 34; but, in the case of very slender columns, it will be necessary to

Mr. Mensch.



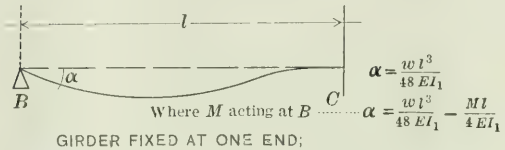
SIMPLE GIRDER UNIFORMLY LOADED.

FIG. 21.



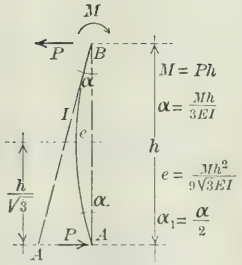
SIMPLE GIRDER, WITH MOMENTS, M, ACTING AT ITS ENDS.

FIG. 22.



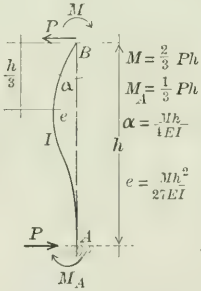
GIRDER FIXED AT ONE END;
SIMPLY SUPPORTED AT THE OTHER END.

FIG. 23.



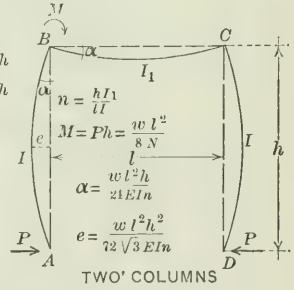
COLUMN AFFECTED BY A MOMENT, M, ON TOP AND FORCES, P, AT THE ENDS
BOTTOM OF COLUMN NOT FIXED.

FIG. 24.



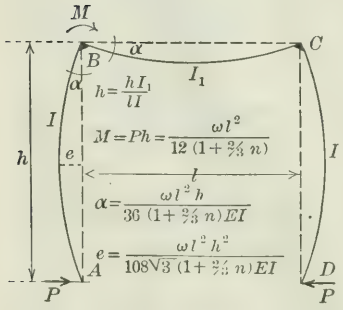
AS IN FIG. 24
BUT BOTTOM OF COLUMN IS FIXED.

FIG. 25.



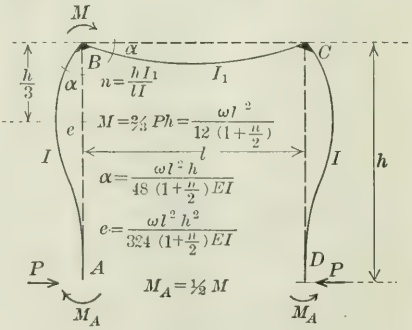
TWO COLUMNS WITH GIRDER RESTING LOOSE ON TOP OF COLUMNS
BOTTOM OF COLUMNS NOT FIXED.

FIG. 26.



TWO COLUMNS,
WELL-CONNECTED WITH GIRDER AT TOP.
BOTTOM OF COLUMNS NOT FIXED.

FIG. 27.



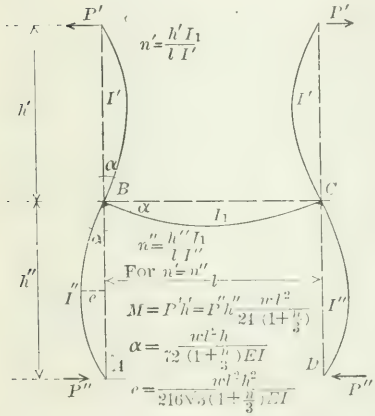
AS IN FIG. 27,
BUT BOTTOM OF COLUMN IS FIXED.

FIG. 28.

find the moment caused by the girder at the point of the greatest deflection of the columns and to combine the eccentricity caused by the moment with the eccentricity caused by the deflection of the column. Mr. Mensch.

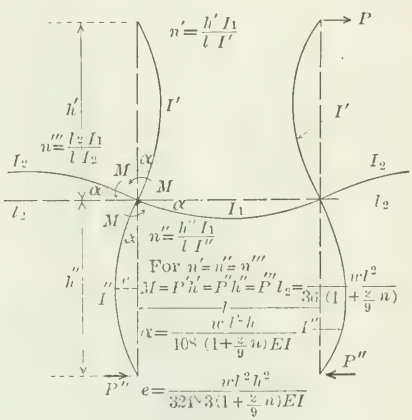
As an example, assume a frame like Fig. 27, consisting of two 8-in. H columns, weighing 32 lb. per ft., 30 ft. high, and well connected with a B-30 Bethlehem I-beam, carrying a load of 100 000 lb. on a span of 40 ft.

The moment of inertia of *B*-30 = 5 240 in.^{4}
" " " " " 8-in. H = 106 "



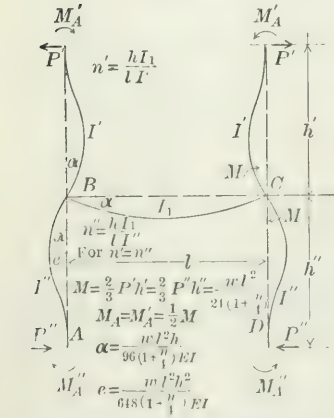
TWO TWO-STORY COLUMNS WITH GIRDER WELL CONNECTED. ENDS OF COLUMNS NOT FIXED.

FIG. 29.



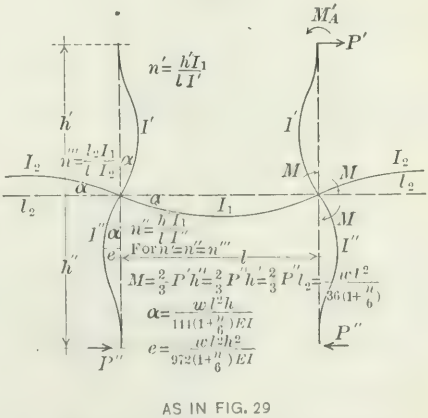
AS IN FIG. 29 BUT THE GIRDER IS CONTINUOUS.

FIG. 30.



AS IN FIG. 29 BUT THE COLUMNS ARE FIXED AT THE ENDS.

FIG. 31.



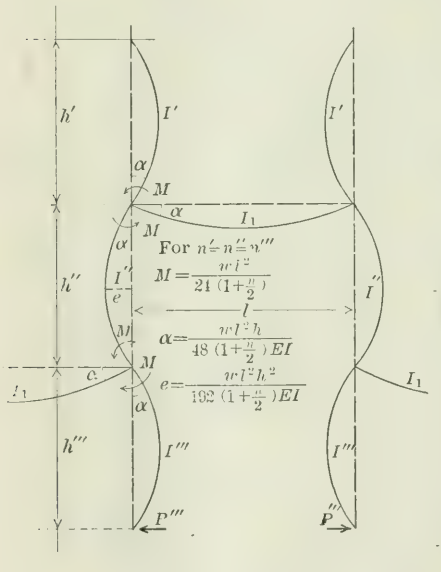
AS IN FIG. 29 BUT THE GIRDERS ARE CONTINUOUS, AND COLUMNS FIXED AT THE ENDS.

FIG. 32.

Mr. Mensch. therefore, in Fig. 27, $n = \frac{30 \times 5\,240}{40 \times 106} = 37.1$, and the moment at the junction of column and girder, as shown in Fig. 27,

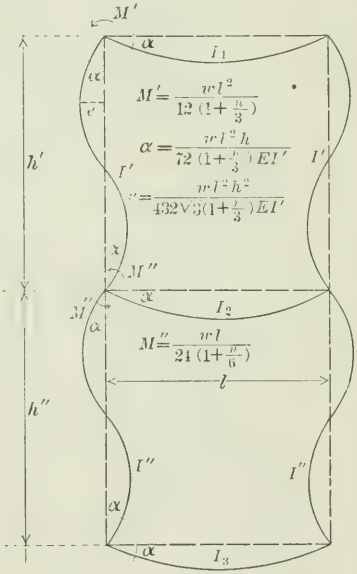
$$M = \frac{100\,000 \times 40}{12\left(1 + \frac{2}{3}\,37.1\right)} = 13\,000 \text{ ft-lb.}$$

If the column is considered as a short compression member affected by this moment, it is found that $e = \frac{13\,000}{50\,000} = 0.26$ ft., or $\frac{e}{d} = \frac{0.26}{0.666} = 0.39$.



TWO THREE-STORY COLUMNS, WITH GIRDERS ON OPPOSITE SIDES OF THE COLUMNS.

FIG. 33.



TWO MANY-STORY COLUMNS, WITH GIRDERS LOADED AT EACH FLOOR.

FIG. 34.

From Columns 14 and 13 of Table 25, it is found that $\frac{e}{d} = 0.391$ corresponds to a percentage of strength of 55. Assuming the U. L. P. as 35 000 lb., the column would fail at an average unit load of $35\,000 \times 0.55 = 19\,250$ lb., or a total load of $19\,250 \times 9.17 = 176\,500$ lb. The actual load being only 50 000 lb., or 5 460 lb. per sq. in., the factor of safety is $\frac{176\,500}{50\,000} = 3.53$.

If it is desired to consider the deflection of the column, one may proceed in the following manner: The greatest deflection will be at

Mr. Mensch.

the center, and, at the time of failure, is given, according to Equation (23), by

$$\frac{e\,k}{d} = \frac{0.0015 \times 30^2}{8 \times 0.67^2} = 0.75 \ldots\ldots\ldots\ldots (26)$$

The moment exerted by the girder in half of the column height is only one-half the moment at the junction, or 6 300 ft-lb., and the corresponding eccentricity,

$$e_1 = \frac{6\,300}{50\,000} = 0.126 \text{ ft., or } \frac{e_1}{d} = \frac{0.126}{0.666} = 0.189 \ldots . (27)$$

We have now to add $\frac{e}{d}$ and $\frac{e_1}{d}$, in order to find the total eccentricity under which the column fails, but this cannot be done at once, because we do not know, *a priori*, the value of k in Equation (26). The addition must be accomplished by trial. Assuming $k = 1$, we find from Equation (26) $\frac{e}{d} = 0.75$, adding $\frac{e_1}{d}$ from Equation (27), the total $\frac{e}{d} = 0.75 + 0.189 = 0.939$. From Columns 14 and 13 of Table 25, it may be seen that the ratio of $\frac{e}{d} = 0.939$ is between the values of $k = 1$ and $k = 0.85$, and is probably very close to $k = 1$. The values of W and $\frac{e}{d}$ are very unsteady in this region, because the tension flange is affected so greatly by a small change in the position of the neutral axis. Inasmuch as $k =$ nearly 1, $\frac{e}{d} = 0.939$ corresponds to an $\frac{e\,k}{d}$ of nearly 0.939, and, by further trial, it is found that $\frac{W}{A\,f_c} = 0.40$. Hence these columns will fail at a load of 35 000 $\times$ 0.40 = 14 000 lb. per sq. in., and the factor of safety will be $\frac{14\,000}{5\,460}$, or 2.56.

The slenderness ratio of these columns (if they are considered as rigidly held for sidewise deflection) is only $\frac{360}{3.6} = 100$, and yet a stress of only 5 430 lb. per sq. in. produced as low a factor of safety as 2.5 in steel having a U. L. P. of 35 000 lb. A careful analysis of the condition under which a column is loaded, therefore, is of vital importance in its design, especially when the factors of safety are as low as the Committee states. The example chosen is not so rare in practice as it may appear; such cases occur in theater construction and garages, also in railroad crossings in the large cities.

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This theory allows also a clearer conception of the behavior of latticed columns. On account of the final eccentricity of the column as a whole, the stresses in the dangerous section will be unequally distributed, and the magnitude of these stresses can be found from the values of k in the tables. According to whether the column is truly hinge-ended or flat-ended, there exist sections, either at the top or at the quarter points, where the average stresses are equal in magnitude on both sides; and, between these points, the difference of the load on the separate members must be transmitted by the lacing. This transmission of the shears is not uniformly distributed, however; it is greatest near the ends or at the points of inflection.

In regard to the column strength of a latticed column, it must be considered that the eccentricity, e, of the column as a whole, is diminished as the square of the inverse ratio of column length to panel length for each panel, and to this eccentricity must be added the $\frac{e\,k}{d}$ of the separate members between the latticing. There are quite a number of fine points to be considered, and these must be proved by tests, before a definite theory of latticed columns can be established.

Steel Column Committee.

STEEL COLUMN COMMITTEE* (by letter).—The discussions on the Final Report of the Special Committee on Steel Columns and Struts, seem to voice a shade of disappointment because the work of the Committee was not carried to a point where the final word could be said on most of the complex questions relating to the design, ultimate strengths, and safe working values of various types of steel columns.

The report, however, should be looked on as a contribution to general knowledge, rather than as the last word on the subject. It records, as it now stands, what is probably one of the greatest engineering investigations in magnitude, cost, and completeness, even though the larger part of the work first contemplated yet remains undone.

Now that the war is over and two years of reconstruction have been successfully passed, the Committee is of the opinion that the investigation of the theory and practice of column design should be made the subject of study by a special committee of the Society.

For the benefit of future investigators, the Committee desires here to record some additional thoughts, as follows:

1.—Broad Range of Programme.—Table 27 indicates the complete programme as originally discussed by the Committee. The work which has been finished is shown by bold-faced type, and represents what the Committee outlined as the first installment and one which would give certain results within a reasonable time. It is evident that much

* Lewis D. Rights, Chairman, C. W. Hudson, Secretary, James H. Edwards Charles F. Loweth, Ralph Modjeski, George F. Swain, and J. R. Worcester. Emil Swensson, M. Am. Soc. C. E., a member of the Special Committee, died on May 13th, 1919.

remains to be done before the subject will be covered comprehensively. It is to be hoped, however, that the work so far accomplished has shed some light on the subject, and that the thoroughness of the investigation will eliminate the necessity of repeating this part of the programme. Steel Column Committee.

TABLE 27.—General Programme for Column Tests.

Type.	End condition.	Grades of steel.	Loading.	Length ratio, $\frac{l}{r}$
(1) **SOLID** (Unstayed sections).	(1) **SQUARE**.....	(1) Soft. (2) Medium. (3) Hard. (4) Special alloys.	(1) **AXIAL**....	**20–50** **85–120** **155–200** **250**
	(2) Hinged (Pin and rocker).		(2) Eccentric.	
	(3) Fixed (Riveted bases).	(5) **SPECIAL.**	(3) Cross-bending.	
	(4) Riveted (Truss member connections).			
(2) Stayed (Lattice and battens).				
(3) Special (Circular).				

Variations.
LIGHT AND HEAVY SECTIONS,
Time test.
Repetition tests,
SPECIAL RIVETING.

Note.—Bold-faced type indicates programme of columns tested.

2.—Variations in Steel.—One of the most important results, in the opinion of the Committee, has been to call attention to the variations in the structural steel in general use. The discussion indicates that many structural engineers are not fully aware even now of this important condition. As substantiating the results of experiments, it is interesting to note the views of metallurgists, two of which are quoted, as follows:

(*a*) In a paper read before the American Iron and Steel Institute, on May 31st, 1918, Theodore W. Robinson, Vice-President of the Illinois Steel Company, says:

"The manufacture of steel is far from being an exact science. It is an art founded upon scientific principles and to a degree subject to scientific control, but there is much in the metallurgy of steel that bespeaks ignorance instead of knowledge, uncertainty instead of definite determination."

Steel Column Committee.

(*b*) In a paper presented before the American Society for Testing Materials (Vol. VI, 1906), Charles L. Huston, a manufacturer of structural steel, stated:

"Under the present standard practice and known methods of manufacture, it is difficult, or nearly impossible, to secure steel that even in one moderate sized plate will have anything like the uniformity generally supposed to exist."

Mr. Huston records very complete tests on a number of ingots, and shows variations in ultimate strengths of the steel, amounting to 19 000 lb. per sq. in. in the same ingot.

It is to be hoped that in the future not only will means be found to control these irregularities, but to allow for them in design.

3.—Conservative Recommendations of the Committee.—It appears to the Committee that the only method of allowing for the irregularities which now occur in strengths of structural steel is to recommend conservative working values. This view has also been shared by the Committee on Iron and Steel Structures of the American Railway Engineering Association in its report for 1920, and by the Joint Committee appointed by the Board of Direction of the Society to confer with a Committee of the American Railway Engineering Association.*

The working formula recommended by the Joint Committee is as follows:

$$P = 15\,000 \text{ lb.} - 50\frac{l}{r},$$

maximum value 12 500 lb., with limiting values of $\frac{l}{r}$.

Although this formula does not agree exactly with the units recommended by the Committee, it indicates that the report of the Committee has had an influence in modifying the formula previously in use.

* *Proceedings*, Am. Soc. C. E., February, 1920, p. 142; also, *Bulletin*, American Railway Engineering Association, January, 1920, p. 431.

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TRANSACTIONS

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Paper No. 1457.

NOTES ON THE GUATEMALA EARTHQUAKES, AND EARTHQUAKE-PROOF CONSTRUCTION

By S. Vilar y Boy, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Marsden Manson, A. W. Buel, and S. Vilar y Boy.

Synopsis.

Up to the present time, there has been no possibility of calculating earthquake strains for practical building purposes, due chiefly to the uncertain knowledge of physical conditions of brickwork and masonry in the destroyed or damaged structures. After the use of reinforced concrete had become general in Guatemala, the effect of earthquakes on such structures led to some conclusions as to the destructive power compared with the transverse load for which the damaged or undamaged buildings had been designed, but not much has been done, to the writer's knowledge, in the way of mathematical analysis.

The damage done by the Guatemala earthquake to some steel structures afforded an opportunity to use it as a starting point for more definite study, as shown in this paper.

The abnormal conditions brought about by those earthquakes, together with the World War, compelled the writer to make some tests of various tropical woods which, rough though they are, led him to certain conclusions as to the nature of the strength of wood, which might be extended to other building materials. It also appears from

the results of these tests, that some relation exists between the brittleness of wood and its durability.

The reconstruction of Guatemala City after the earthquakes that practically destroyed it in December, 1917, and January, 1918, together with the difficulties of importing building materials on account of the European War, brought up that question mentioned by Rankine as defining the nature of most engineering problems, "What to do rather than what to think, in order to start on the rebuilding work without delay."

It was necessary therefore to investigate practical and reasonable solutions, disregarding elaborate investigations, and from this point of view the writer endeavored to obtain the necessary data to design several buildings for the International Railways of Central America, with which he is connected, as well as others for private parties who called on him for that purpose.

The first point to be determined was of course the force of the earthquake in order to design the structures to stand for an equal shake with the customary factors of safety. This was quite uncertain since all wrecked buildings had been constructed of materials such as raw brick or adobe, brickwork or common masonry, the physical properties of which vary within wide limits; also because buildings of the same type of construction did, or did not, collapse, depending on their location.

As a starting point, the writer had the records published by the Society on the San Francisco earthquake,* stating that none of the concrete buildings designed for a wind pressure of 30 lb. per sq. ft. had collapsed; also a report on the earthquake in Sicily by the Royal Italian Committee estimating the earthquake impact as equivalent to one-twelfth of the weight of the structure for the ground floor and one-eighth from the second story up. It was necessary, however, to make sure that the Guatemala earthquake had not been stronger than those just mentioned.

The earthquake impact is a force of inertia depending on mass (or weight) and acceleration, the acceleration being, more or less, dependent on the manner in which the structures are supported, as

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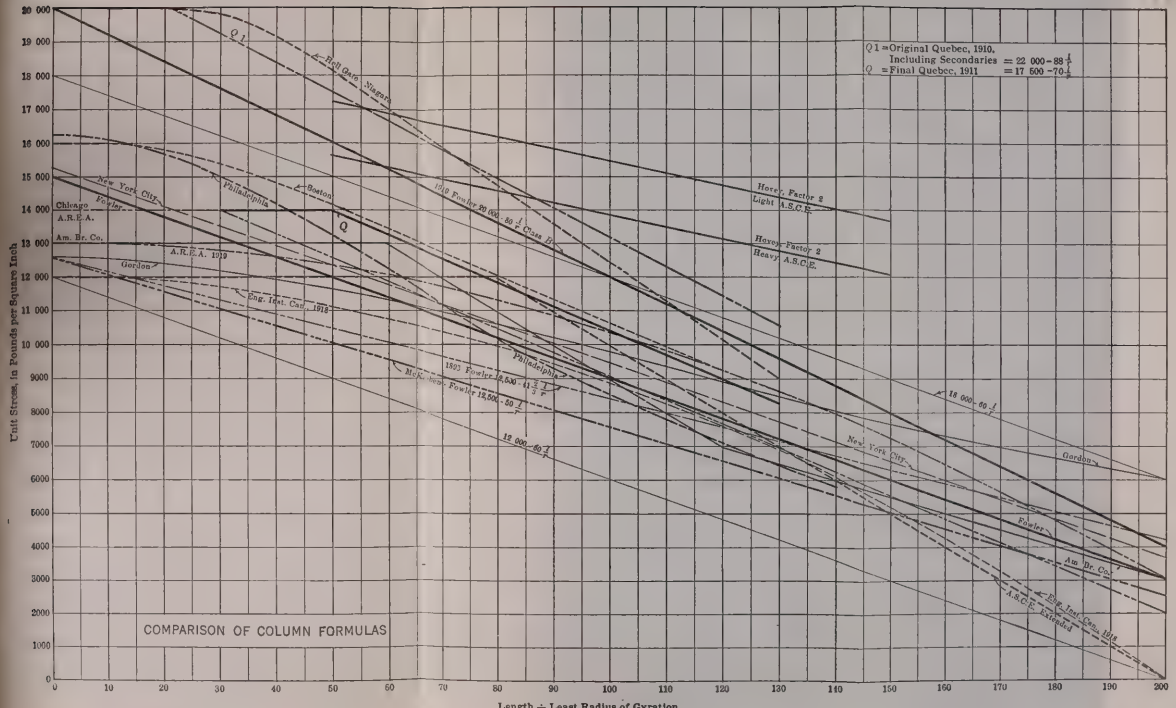
* *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 208.


Q 1 = Original Quebec, 1910, Including Secondaries = 22 000 − 88 l/r
Q = Final Quebec, 1911 = 17 500 − 70 l/r
Unit Stress, in Pounds per Square Inch
Length ÷ Least Radius of Gyration
COMPARISON OF COLUMN FORMULAS
Q 1
Q
Tell Gate, Niagara
Boston
Philadelphia
New York City
Chicago
Fowler
A.R.E.A.
Am. Br. Co.
A.R.E.A. 1919
Gordon
Eng. Inst. Can. 1918
1910 Fowler 20 000 − 80 l/r Chain D.
Hovey, Factor 2
Light A.S.C.E.
Hovey, Factor 2
Heavy A.S.C.E.
1893 Fowler 18 500 − 41 3/4 l/r
M.C. Hovey, Fowler 18 500 − 50 l/r
12 000 − 60 l/r
18 000 − 69 l/r
New York City
Gordon
Fowler
Am. Br. Co.
A.S.C.E. Extended
Eng. Inst. Can. 1918

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in the theory of impact. That is to say, the earthquake is more destructive on bodies likely to vibrate freely than on structures properly braced.

Bearing these principles in mind, the writer tried to get some instructive data from the ruins of Guatemala City. The only specimens having definite physical properties that could be found were the electric light posts, made of rolled steel I-beams, some of which were bent by the earthquake force. They were strained, therefore, beyond their elastic limit or yield point. Since some did bend and some did not, in the same neighborhood, it follows that they were not strained much beyond the average yield point of steel; otherwise, all should have bent.

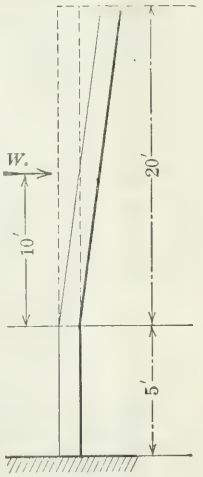

FIG. 1.

Those columns, buried in the ground, were 25 ft. high above the surface and the bend was about 5 ft. above the ground line, as in Fig. 1. The static load, W, equivalent to the earthquake impact, applied at the center of gravity of the bent portion therefore can be obtained as follows, S being the section modulus:

$$W \times 120 = 30\,000\ S.$$

For a $7\frac{1}{2}$-in. I-beam with $3\frac{1}{4}$-in. flange width and $\frac{3}{8}$ in. thick, S was taken as 12. Therefore, $W = 3\,000$ lb.

This could not be taken as an absolute value for any pole of the same height, but should be considered as a force of inertia for the weight of the bent portion of the beam, or 330 lb. (16.5 lb. per lin. ft. for the 20-ft. length). Hence:

$$\frac{\text{Earthquake Impact}}{\text{Weight of Column}} = \frac{3\,000}{330} = 9.1$$

Or, say, 9, in round numbers, for a pole buried in the ground, free at the top, and acting as a cantilever.

This reasoning was satisfactorily checked by the formula for the acceleration of an earthquake given in the Encyclopedia Britannica, the writer thinks on Milne's authority, as follows:

$$a = \frac{1}{6}\,\frac{g\ F\ A\ B}{f\ w}$$

in which

a = acceleration per second per second;

F = ultimate strength, or force per unit surface, which when gradually applied produces fracture;

A = area of base fractured;

B = thickness of column;

f = height of center of gravity of column above the fractured base;

w = weight of the portion broken off;

By substituting and calling x the breadth of a rectangular beam, $A = x\ B$, and

$$\frac{A\ B}{6} = \frac{x\ B^2}{6} = S$$

where S is the section modulus for a rectangular beam. Then

$$a = g\ \frac{F\ S}{f\ w}$$

and

$$m\ a = \frac{a\ w}{g} = \frac{F\ S}{f}$$

or, force of inertia $\times f = F \times S$.

In other words: Bending moment due to earthquake impact must equal resisting moment. This coincides exactly with the writer's point of view, and the foregoing computations. These computations apply to a pole with a free end acting as a cantilever under the earthquake transverse force, whereas the walls of a building are tied across by joists or roof trusses which make them act as supported beams. In fact, the pillars and columns with flat ends are about an intermediate case between supported and fixed ends, but if they are dealt with as supported ends the results will be on the safe side.

The likelihood of vibration, for different beams, is measured by the deflection taken under similar loading conditions, and, therefore, the deflection ratio will show the ratio of the force of inertia on a cantilever and a simple beam of the same weight, cross-section, and span.

For the sake of comparison, the writer adopted the formulas for deflections of beams under uniformly distributed loads, although the distribution of forces of inertia along the beam is not uniform. In accordance with this principle, if the earthquake impact on a canti-

lever is 9.1 times its weight, the same earthquake impact on a simple beam should be:

$$9.1 \times \frac{8}{76} = \frac{9.1}{9.5} = 0.95 \text{ times its weight.}$$

Although the assumption of simply supported ends for columns and pillars of buildings, as made in the foregoing computation, results on the safe side against the error due to the non-uniform distribution of forces of inertia along the vibrating beams, the theory was not accepted until checked by facts, the necessary confirmation having been found in the damage done by the earthquake to Las Vacas Viaduct, 3 miles east of Guatemala City. This damage consisted in the shearing of some of the sole-plate rivets connecting the trusses to the towers, although none of the trusses was displaced from the supports.

The Las Vacas Viaduct is 743 ft. long, 203 ft. in maximum height, and its weight is 976 tons of structural steel, with 166 tons of track material, or a total of 1 142 tons. There are seven towers, the damaged ones being 127 ft. high, which is practically the average.

It might be assumed, therefore, that the weight of the part of the structure straining the sole-plate rivets was one-seventh of the total weight, or 163 tons. There are sixteen sole-plate rivets, $\frac{3}{4}$ in. in diameter, on a tower, with sole-plates $\frac{7}{16}$ in. thick. The ultimate strength of these rivets should be 157 tons at the rate of 30 tons per sq. in. The earthquake impact, according to the foregoing theory, should be $0.95 \times 163 = 155$ tons. It is seen that these two figures practically agree.

Again, since only two towers were damaged, it follows that the strain was not much beyond the breaking point; otherwise, all the forces should have given way. Also, in all probability there were along the viaduct some fixed points, or noduli, as there are along vibrating strings.

As to the analysis of the nature of the strains developed by the earthquake, the writer did not even attempt to undertake it, since such strains are of all kinds, oscillation, trepidation, and swirls.

The foregoing refers to the reinforced concrete or steel frame of the buildings, the panel walls of which are brickwork in many cases—the only feasible type of construction in view of the scarcity of cement and steel due to the war. It was quite difficult to determine the earthquake impact on brickwork panels, since the application of the elastic theory

to such a variable material as brickwork is rather uncertain, but for a comparison with other observations they were considered as fixed slabs, since they are held on four sides by concrete pillars and wall-plates. To obtain a simple formula for the deflection of such brick panels, in order to estimate the corresponding earthquake impact, the following theory was applied. From the fundamental equation,

$$\frac{E\,I}{\rho} = M$$

the deflections are proportional to the moments. In ordinary practice it is accepted that the bending moment for a square concrete slab is $M = \frac{W\,l}{20}$, as compared with $M = \frac{W\,l}{10}$ for a beam. That is to say, the bending moment for the slab is half the bending moment for the beam under the same load.

It appears, therefore, to be close enough to assume that the deflection, Δ, for the slab should be half the deflection for a beam with ends fixed, or, say,

$$\Delta = \frac{P\,l^3}{768\,E\,I}.$$

If so, the ratio of the earthquake impact to the weight should be, by comparison with the ratio for a cantilever as obtained in the foregoing, as follows:

$$9.1 \times \frac{.8}{768} = 0.09.$$

This agrees quite well with the ratio $\frac{1}{12} = 0.83$, given by the Italian Committee.

A 24-in. wall, assuming its weight to be 150 lb. per cu. ft., should have an earthquake impact per square foot of 25 lb. according to the ratio given by the Italian Committee, or 27 lb. according to the writer's assumptions. Adding a wind pressure of 30 lb. per sq. ft., the distance between the concrete pillars required to prevent the brick panels from collapsing can be computed.

This was done by considering a portion of wall 1 ft. deep, disregarding the slab effect so as to be on the safe side. This section of wall, assumed as a simple beam, would be acted upon by an earthquake impact 0.95 times its weight, and the stress should not exceed the allowable transverse stress, taken as 10 lb. per sq. in. as a conservative value. The

end shear should not go beyond 2 880 lb. and the corresponding weight of wall for half the span should not be more than $\frac{2\,880}{0.95} = 3\,030$ lb. Assuming the weight of 1 lin. ft. of wall, 2 ft. thick, at 300 lb., it follows that a 20-ft. distance between pillars is perfectly safe.

It might be objected that the weight of roof has been neglected in the previously mentioned calculations, but the writer thinks that such weight rather adds to the stability of the walls by bracing them against each other.

As a matter of fact, the inspection of various ruins proved that, in almost all cases, the gable ends of buildings collapsed while the side-walls supporting the roof trusses remained standing in more or less bad shape, which goes to prove that the roof trusses were more of a help than a weakness, in spite of their weight, which, by the way, was rather great, since before the earthquake practically all the roofs in town were of pantiles.

Finally, sufficient reinforcement was allowed in the concrete wall-plates to make a belt around the buildings and main partitions, the tensile strength of which should prevent cracking at the corners, this having been suggested by S. F. Shaw, M. Am. Soc. C. E., at that time Chief Engineer of the International Railways of Central America.

Although the preceding reasoning and calculations are far from accurate, they agree well with facts, and they were developed with the idea of being on the safe side and within reasonable limits.

Tests of Native Woods.

In connection with the reconstruction work, some tests were made of several kinds of wood which are not commonly used for building purposes, with a view to adopting those passing the tests satisfactorily. The writer thinks the results of his investigation might be of use to engineers in tropical countries, if not as absolute values, at least as a guide.

The investigations were based on some fundamental ideas pointed out in the discussion of a paper by Henry S. Prichard, M. Am. Soc. C. E., on "Faults in the Theory of Flexure",* that is, the transverse deformation of materials depends not only on the strength of fibers themselves, but on the bond between fibers as well, especially in wood,

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 895.

the fibers of which adhere to each other by means of such a variety of cementing substances. Furthermore, some connection between the elasticity of timbers and their durability was sought as a check to supplement the chemical treatment, as explained later.

Among cementing substances, the albuminoids and carbo-hydrates are food for fungi and insects. It was the writer's opinion that all kinds of wood with a high modulus of elasticity, E, as compared with ultimate transverse strength or modulus of rupture, R, which ratio might be called "modulus of brittleness," would contain a large quantity of albuminoids and sugar, and hence attract fungi and insects and cause rapid weakening and decay; also, that there should be some more or less constant ratio between the durability in the ground and the resistance to artificial decay by chemicals. The only chemical available at that time for applying the latter test was the hypo-sulphite of soda commonly used in photography, which was substituted for the sulphite of lime to make paper paste.

By comparing the behavior of some known and some relatively unknown kinds of wood through these tests, the writer expected to arrive at some reasonable conclusions in regard to durability. The rough means of experimenting available at the time make the figures, given later, unsuitable for accurate calculations, but approximate enough as a guide.

The transverse bending strength, R, and the coefficient of elasticity, E, were determined by simply supporting a beam 1 in. square on supports 24 in. apart, the center breaking load consisting of old castings and track bolts, the deflections being measured by a thread stretched between supports. For this case, the ordinary theory of flexure leads to the following expressions:

R = ultimate strength in bending = $36 \times$ center breaking load

$$E = 3\,456 \times \frac{\text{Center load}}{\text{Deflection, in inches}}.$$

Four specimens of each kind of wood were tested, the deflections being measured under five different loads for the same specimen. The timbers were unseasoned; in fact, they were samples sent by the proposed contractors.

The chemical treatment was applied by putting a slice, $\frac{1}{4}$ in. thick, of the broken 1-in. beams in a vessel made of a 3-in. nipple closed with

pipe plugs at the ends, the vessel being filled with 75 cu. cm. of a solution of hypo-sulphite of soda with a strength of 36% and 3 cu. cm. of hydro-chloric acid with a density of 1.08. These vessels were enclosed in a cylinder connected to the shop boiler and fitted with a steam gauge to maintain the unit pressure at 45 lb. during 45 hours.

After treatment, the specimens were tested with a home-made Vicat needle, 1 mm. square, weighing 5½ lb., the penetration of which afforded a comparison between the known and relatively unknown woods.

Each specimen was tested ten times. Most of the native woods that were tested have never been classified, and it is impossible, therefore, to give their scientific names, but engineers who have worked in the tropics will very likely recognize many of them. By comparing the results obtained for the known woods, it was considered that values of the modulus of brittleness under 150 are favorable, as a rule, but for very strong woods, the ultimate transverse strength of which is more than 13 000 lb. per sq. in., values of the modulus of brittleness might be acceptable up to 200.

A needle penetration of less than 2 mm. appeared to be favorable. The strength of chemicals and time of steaming were determined by trial.

The results of the tests are given in Table 1.

TABLE 1.—Results of Tests of Tropical Woods.

Sample No.	Name of wood.	Modulus of rupture. R, in pounds per square inch.	Modulus of elasticity, E, in pounds per square inch.	Modulus of brittleness, $\frac{E}{R}$	Size of nail splitting specimen.	Needle penetration after treatment, in millimeters.
1..	Sangria.....	12 736	1 558 000	122	40d.	1.47
2..	Sangre	5 652	1 261 000	223	40d.	6.00
3..	Leche Amarilla ...	10 605	1 819 000	171	20d.	2.30
4..	Santa Maria.....	10 449	1 539 000	147	20d.	1.47
5..	False Fustic.....	12 663	1 862 000	148	20d.	2.30
6..	Fustic.....	14 751	2 702 000	186	16d.	Practically nil.
7..	Hormigo	9 847	1 705 000	173	20d.	1.75
8..	Red chichique.....	11 754	1 857 000	158	20d.	2.20
9..	White chichique...	17 335	4 848 000	279	20d.	6.00
10..	Chichipate	21 474	2 509 000	117	20d.	Practically nil.
11..	Nance	13 167	2 408 000	182	20d.	1.10
12..	Native pitch pine..	8 766	1 582 000	180	30d.	2.40
13..	Campanillo.....	16 380	2 339 000	142	16d.	1.30
14..	Güachipilin.....	16 587	2 095 000	131	20d.	0.70
15..	Genizaro.....	10 953	1 569 000	144	20d.	1.00
16..	Laurel (Atlantic) .	6 644	958 000	145	40d.	5.00
17..	Black laurel.....	10 278	1 239 000	120	20d.	0.60
18..	Madrecacao	12 285	1 556 000	126	20d.	0.40
19..	Güayacan.....	.....	.....	...	....	Practically nil.

Sample No. 1, Sangria.—There were contradictory reports about this sample, Sangria, because of different woods with similar names.

Sample No. 2, Sangre.—This is very similar to Sample No. 1, but of a lighter weight and of the undesirable kind; the results of the tests made it possible clearly to differentiate both kinds.

Sample No. 3, Leche Amarilla.—On account of its yellow juice, the third sample had the name "Leche Amarilla". The ultimate transverse strength 10 605 lb. per sq. in. is the average of four specimens, one of which was as low as 7 000 lb.

Sample No. 4, Santa Maria.—Some foundation blocks of Santa Maria were reported to be perfectly sound after five years in the ground.

Sample No. 5, False Mora, or Fustic.—A specimen of an odd wood was received, the sender claiming that it was Mora or Fustic which during the war was exported as dyewood. The results of the tests are as given in Table 1. The needle penetration of more than 2 mm. and the modulus of brittleness under 150, but very close to it, contradicted the well-known lasting qualities of Fustic. This being the case, the original log was examined by a wood expert and compared with a log of true Fustic, with the result that it was found that the sample was not true Fustic.

Sample No. 6, True Mora, or Fustic.—The results of tests of true Mora or Fustic were as stated in Table 1. The needle penetration was inappreciable and the specimen could not be broken with the fingers as had been done with more or less difficulty with previous samples. This harmonizes with the writer's assumptions and the well-known durability of Fustic, but leads to allowing values up to 200 for $\frac{E}{R}$, if R is more than 13 000.

Sample No. 7, Hormigo.—The sapwood of Hormigo rots out very quickly, although its heartwood has proved satisfactory in actual practice. The specimen used had a fair percentage of sapwood. The modulus of brittleness was rather high, due to the sapwood in the sample, whereas the slice used for chemical treatment was practically all heartwood.

Sample No. 8, Red Chichique; Sample No. 9, White Chichique.—There are two varieties of Chichique, red and white, which lead to contradictory reports. The red variety has a life of, say, 5 years in the ground. The unfavorable results for white Chichique are confirmed by

actual practice, as the white variety is rotten after being 2 years in the ground.

Sample No. 10, Chichipate.—This wood has a very good reputation in actual practice. This specimen was rather well seasoned, as it came from car-shop stock.

Sample No. 11, Nance.—This was reported as giving satisfaction in the ground for foundation blocks.

Sample No. 12, Native Pitch Pine.—The penetration was quite variable, sometimes being as much as 3.5 mm. There are reports of some posts of this quality rotting after 2 years in the ground.

Sample No. 13, Campanillo.—This wood has proved to be fairly good in actual practice.

Sample No. 14, Güachipilin.—This wood is very good in actual practice.

Sample No. 15, Genizaro.—This is fairly good in actual practice.

Sample No. 16, Laurel (Atlantic Side).—There is a very good wood called black laurel on the Pacific Coast, and some samples were brought from the Atlantic Coast with the expectation that they would be as good as the others. The tests showed, however, when compared with Sample No. 17, that this expectation was not fulfilled.

Sample No. 17, Black Laurel.—The results for this wood confirmed its lasting qualities and are quite different from the laurel on the Atlantic side, Sample No. 16.

Sample No. 18, Madrecacao.—The results for this wood require no comment.

Sample No. 19, Güayacan (Lignum Vitæ).—No sample of the required dimensions for the physical tests was available, but for the sake of confirming the effectiveness of the chemical treatment, a small sample on hand was subjected to it. The needle penetration was nil.

Several specimens of unknown woods were tested, as well, and by comparison with the results for the samples as listed in Table 1, some reasonable probabilities were obtained as to their lasting qualities, which made it possible to accept or reject them in contracts.

The foregoing tests were made more than a year ago, but sufficient time has not yet elapsed for actual experience to confirm or contradict the results, although they agree fairly well with the information available at the time.

DISCUSSION

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MARSDEN MANSON,* M. AM. SOC. C. E. (by letter).—This contribution on the Guatemala earthquake and earthquake-proof construction is a valuable addition to the professional studies and literature on this subject.

At best, the efforts to determine the strains produced by earthquakes are as much empirical as analytical. In the San Francisco earthquake of April, 1906, three chimneys in the same building were twisted and displaced laterally at the roof plane in different directions and to variable distances, showing that each yielded to forces applied in different directions and at different times, and that a certain joint or joints which yielded to the first displacing and rupturing strain was near other joints which were unaffected and which yielded to subsequent strains in a different direction. The same was true in failures of other types.

The writer was Chairman of the Sub-Committee on Statistics, of the Reconstruction Committee, and in that capacity investigated several thousand buildings in San Francisco, some within and some beyond the burned area. From the report then rendered the following extracts are taken:

"In presenting data, the Sub-Committee feels justified in adding general remarks and suggestions based on its observations. The suggestions and recommendations are submitted with the hope that they will be fully considered by owners, and that they may result in the most scrupulous regard of all provisions of the architect and engineer when stability and safety are concerned, whether as regards fire or earthquake. With the lessons before us, it is the unanimous opinion of your Sub-Committee that the greater portion of the loss sustained could have been prevented by a clear interpretation and regard for the lessons of 1868 and the fires which have devastated this and other cities.

"Even in the destruction of water mains it is apparent that with mains constructed on solid ground or adequate foundations over critical ground and with ample service reservoirs and fire cisterns, and with well-equipped fire-tug service having shore connections to mains, our disastrous losses of hundreds of millions of dollars would have been measured by tens of millions and probably by a less sum.

"*Small Frame Buildings, Wooden Underpinning.*—In general, these buildings suffered severely owing to the rotten condition or original weakness of the underpinning. The mudsills and adjacent parts of posts and braces had, in many instances, rotted, and the buildings settled vertically, as in a row of frame buildings on the west side of Webster Street, between Eddy and Turk Streets; or they settled laterally, as in the case of one-story frame buildings on the southeast corner of Bush and Steiner Streets, and on the south side of O'Farrell near Devisadero Street. In most instances of this kind very flimsy construction was originally permitted. The damage to the upper parts of these buildings was at times so severe as to totally wreck the buildings.

* San Francisco, Cal.

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"Small Frame Buildings, Masonry Foundations.—These stood well, the damages being generally restricted to loss of chimneys and consequent injuries to roofs, walls, and gutters.

"Large Frame Buildings, Wooden Underpinning.—In general, a stronger original construction and broader base enabled these structures to stand well. Damages to roofs and sides by falling of chimney tops from the building itself or from adjacent buildings constituted the major portion of the damage. In some instances, the roofs were not able to withstand massive chimneys with high fall.

"When masonry foundations were provided, these buildings suffered less, and numerous instances were observed where no damage whatever was done, notably the three-story frame building on the northeast corner of Washington and Gough Streets, where the chimneys, by reason of first-class construction, remained intact, and the building itself shows no evidence of strain. It may be here observed that buildings of this type, well constructed, diagonally boarded and braced, metal or wire lathed, and with chimneys of reinforced concrete or brick, well laid in cement mortar, are perfectly safe in a severer shock than that of April 18th, 1906.

"Class 'C', Brick Walls, Wooden Partitions, Floors, and Posts.—This class of structure has quite a variety of applications, being used for dwellings, stores, storehouses, churches, libraries, etc. When properly constructed, they stood the shock well, particularly if the walls were tied and bonded and the mortar good. Some, however, had two fatal defects—weak or badly laid mortar and unbraced gable ends. Churches, large storehouses, carhouses, barns, etc., were injured, the gable ends being especially weak. This defect has been brought out prominently in this, as well as in higher classes of buildings. Notable instances are on the northeast corner of California and Webster Streets, the south end of the Ferry Building (Class 'A'). Numerous other instances were observed, especially where lime mortar had been used, not at all or but slightly strengthened by the addition of Portland cement; and where fairly good mortar had been badly used or the bricks laid dry, instead of soaking wet, as they should be in good work.

"A marked instance of failure from poor mortar is on the south side of Geary Street, near Fillmore, where well bonded brickwork fell to pieces, and stone veneer, without bond, scaled off the lower story. This scaling is frequently the case in the pressed Roman brick used as facing, without bond, or fastened against wooden buildings with weak metal ties. Failures from these causes have been so general that they have caused universal comment, and it is hoped that the lessons will result in the thorough testing of all cements and other materials and their proper use, as a matter of safety not only to the owner, but to the public in general.

"Equally notable cases of the standing of well-built structures are frequently noticed. The U. S. Appraisers' Building on Sansome, Washington, and Jackson Streets, although partly on made ground, did not suffer the slightest injury, due to the excellent quality of mortar and good workmanship. A seven-story building on the south side of Eddy Street, 100 ft. east of Jones, although subsequently burned, its walls of well-bonded brick, with good mortar, stand intact. Other instances are

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the Jewish Tabernacle on the north side of Sutter Street, between Stockton and Powell Streets, the church on the northwest corner of Fillmore and Jackson Streets, and the one on the southeast corner of Fillmore and Fell Streets.

"Class 'B', Brick or Masonry Walls, Steel or Cast-Iron Posts, and Steel Girders Protected Against Fire, Metal-Lathed Partitions, etc.— The damage to this class of buildings by earthquake was, in general, light; no instances of well-constructed buildings of this type were observed which suffered more than a small percentage of damage. A notable instance is the reinforced and well-built walls of the Palace Hotel, which withstood both fire and earthquake.

"Class 'A', Steel Frame, Non-Combustible Floors and Partitions.— The thoroughness with which these structures withstood the earthquake has been so frequently recorded that it is not necessary for the Committee to note further facts. The most severe injury was to the slim and heavy tower of the Ferry Building, and the south unsupported gable end. This structure stands entirely on piles, 80 to 90 ft. long, driven into harbor mud some 15 to 20 ft. below city base. Yet it withstood the shock, with the exceptions above mentioned, and a few minor injuries to marble wainscoting. Another instance of damage to this class is the north wall of the building at Sacramento and Mason Streets. Buildings of this class, in process of construction, were injured where mortar had not yet set, but hardly a joint was sprung in those which were caught with the frames partly constructed.

"With corrections as to fire coverings of posts and beams and the use of metal frames for doors and windows, with wire glass and proper window protection, it has been demonstrated that this class of buildings will withstand both fire and earthquake, and, with a local supply of water, can be made absolutely safe, even in a general conflagration. But no building in which combustible material is stored will escape a general conflagration, unless an adequate water supply and men to use it are available. Inferior buildings, under these conditions, were saved, and superb Class 'A' buildings, without water, were totally destroyed. This lesson cannot be too thoroughly impressed upon owners and officials.

"Effects of Earthquake on High Stacks and Chimneys.—These structures were generally severely damaged or broken. They are of several cross-sections, circular, square, polygonal, and intersecting squares. Those having square sections appeared to stand best. The percentage of stacks escaping injury is so small that your Committee recommends that in the future these be entirely of metal or of heavily reinforced concrete or reinforced brickwork; in these two latter cases that only cement mortar of the highest strength be used.

"Chimneys.—Chimneys suffered proportionately more severely than anything else. This was due to three causes: (1) worthless mortar; (2) bad workmanship; and (3) unusual strains at the joints of roofs with the chimneys where weakened further by flashings. To overcome these defects will be simple but will require good materials and first-class workmanship. Chimneys stood where well constructed, proving that well selected materials and good workmanship are proof against severer shocks than that lately experienced. To accomplish this result, your Committee recommends: First, that either lime mortar of stand-

ard proportions reinforced with one-third Portland cement, or Portland cement mortar be used with reinforcing steel rods at roof joint and above; or, second, that reinforced concrete of the best quality be used. Mr. Manson.

"Flue linings have resulted in placing reliance upon these, and using flimsy brickwork on the outside. Brick chimneys properly built are safe and strong under any conditions yet experienced, and the linings serve as a subterfuge to lessen the cost and efficiency of one of the best materials of which chimneys can be built, namely, good brickwork.

"*Mortar.*—The results warrant the absolute rejection in the future of lime mortar unless well seasoned and reinforced with Portland cement, and the most stringent supervision should be had over the testing and use of this latter. Inferior cement and poor workmanship will make weak structures ready to fail when the test of actual strain or use comes. Every yard of concrete and every brick laid in the future should be mixed and laid with scrupulous care. Concrete can be made as worthless as much of the brickwork was in this city. Hence it must be made of the best materials, properly mixed, laid and rammed to secure good results; and bricks must be sound and well made, carefully laid when soaking wet in the best lime-cement or cement mortar. These classes of work will insure safety in earthquake shocks of greater severity than that experienced in April, 1906, and with the added security of an ample water supply, proper protection of posts and girders, and guarded openings to all masonry buildings, will make such general and widespread disasters as that following the morning of April 18th, 1906, absolutely impossible.

"*Retaining Walls.*—Retaining walls have long been a source of expense and danger in San Francisco, the percentage of these failing from various causes has always been large. In a city of such variations of level they are necessarily much used. Both ordinary and surcharged walls failed from lack of sufficient gravity section and inadequate footings. An instance of both these deficiencies is in the overturned surcharged wall on the west side of Gough Street, between Pacific Street and Broadway. Instances of each were noted. As examples of well-constructed retaining walls, the Committee notes those on Pine, Powell, and Mason Streets, and on the south side of California Street, west of Presidio Avenue, which, with others, withstood the shock without injury, proving that properly constructed retaining walls can be relied upon with confidence.

"*Monuments and Statuary.*—These slender structures withstood the shock well. In cemeteries, some were overthrown. The Dewey Monument in Union Square was reported 'standing at an angle of 10 degrees'. An inspection shows that the sections were barely displaced and the pedestal very slightly spalled at a few joints, and that, to the ordinary eye, the monument stands unscathed by either earthquake or fire. Monuments and groups of statuary in other portions of the city were uninjured.

"*Conclusion.*—The Sub-Committee on Statistics finds:

"(1).—That well-built structures of all classes generally withstood the earthquake without serious injury, and that the majority of these structures were unharmed.

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"(2).—That the principal damage by earthquake was due to inferior workmanship, worthless mortar, or faulty design. As instance the City Hall, in which the design of the building is radically unsuited both to the requirements and to the strains likely to occur. Immense gable ends and cornices resting upon massive columns, not well tied to the remainder of the building, were the cause of damage to a far greater extent than inferior mortar and workmanship, as has been alleged. The fragments of masonry lie in large masses rather than in individual bricks, as in many other fallen walls. The same is true of the church on Bush and Pierce Streets, and other instances. These are deficiencies which can be remedied in the future.

"(3).—That losses by fire were in the greater part due: (*a*) to the utter failure of the pipe system, as made manifest by the extinguishment of fires where a meager water supply was available; (*b*) to lack of protection to openings in masonry buildings, and, in the burned buildings of Classes 'B' and 'A', to the additional lack of adequate fire coverings for posts and girders, causing collapse of these members in Class 'B' and in some instances in Class 'A' buildings. These are also defects which can be remedied so as to lessen insurance risks and rates and ultimate damage in case the contents of these structures should burn.

"(4).—That Class 'C' buildings constitute the poorest and most dangerous structures, and should be restricted in use, steel framed and galvanized iron sides and roofs being preferable. Also, that a greater use should be made of non-combustible materials, steel, concrete, galvanized iron, metal lathing, etc., than the present fire and building laws admit. This will be to the advantage of owners and the greater safety of the city.

"(5).—The members of this Sub-Committee feel that too great importance cannot be laid upon the thorough inspection and testing of cement and other building materials and upon the most conscientious workmanship, and are assured that craftsmen of all branches cannot render a greater service to the community nor to themselves than by refusing to do work of any kind which is not up to a high standard of workmanship; for neither themselves, their families, nor any one else will be safe, nor property secure, with such flimsy work as has characterized many buildings of all classes of structures in this city.

"Streets and lots graded with waste and loose materials of all kinds are not suitable foundations on which to make sewers and pavements nor erect homes or buildings for human occupation and use, nor are sewers and buildings with inferior mortar and poorly laid brick on filled areas of this kind more likely to stand in the future than in the past.

"The causes of our great losses may therefore be briefly stated: (1), Failure of water supply due to insufficient number of service reservoirs and cisterns, and to fractures of the mains and connections where constructed over made ground and marshes and over or immediately adjacent to faults—mains should be altogether on solid ground, or on the heaviest of masonry and pile foundations, and service reservoirs and cisterns should be more numerous and larger; (2), Flimsy construction, and in some instances, improper design; (3), Absence of

adequate door and window protection for masonry buildings; (4), Lack of efficient covering for posts and girders in Classes 'B' and 'A'. Mr. Manson.

"The necessity for constructing buildings of incombustible materials has been made fully apparent. These buildings must meet all requirements, from the costly and elaborate steel cage, concrete walled, and floored structure with metal doors, window frames, casings and finishings, to the cheap steel-framed shed, covered and roofed with galvanized iron. To meet these requirements, the following sub-classes of Class 'A' are suggested:

"Class 'A'-1.—Steel-cage construction, concrete, brick or tile walls and floors; metal-framed doors, windows and finishings; plate wire glass windows, asbestos curtains inside.

"Class 'A'-2.—Concrete or brick walls; steel posts concrete or tile-covered; concrete steel or tile floors; steel-framed, metal-covered roof, fire shutters.

"Class 'A'-3.—Reinforced concrete walls; steel-framed roof, metal covered.

"Class 'A'-4.—Light steel frame; walls, metal lathing or expanded metal, plastered with cement mortar on one or both sides of framework; concrete, steel or tile floors; metal roof; metal-framed windows and doors; plate wire glass; asbestos curtains, for light stores or dwellings, etc.

"Class 'A'-5.—Steel frame, corrugated iron sides and roof for sheds and lightest buildings; if more than one story, upper floors of steel and concrete or tiles.

"Class 'A'-6.—Any combination of the above classes which is structurally sound.

"It will be observed that each of these classes is composed of incombustible materials, that they vary in cost from the costly skyscraper to a cheap steel-framed, galvanized-iron shed, and will thus meet the wide requirements of cost, and at the same time afford a reasonable but imperative safety. The mortar to be either lime cement mortar or cement mortar.

"If the radical deficiencies above outlined be remedied, San Francisco will have but little to dread from even a more serious earthquake than that of April 18th, 1906; and if, in rebuilding, we make stability and thoroughness the prime essentials of all work, these characteristics will mark our recovery and development in every line to the remotest future."

The writer is of the opinion, after personal inspection, that most of the loss of property and life in recent earthquakes in Italy and much of that throughout Mexico, was due to the use of weak lime mortar and that thinner walls of good masonry or, better, of reinforced concrete with greater resilience and the use of well-framed timber buildings on well-laid masonry foundations would have greatly lessened the losses; and that these requirements should be prescribed in regions subject to earthquake.

The unanimous opinion of the Sub-Committee sustained the views expressed in the quoted portions of the report. The Sub-Committee

Mr. Manson. was composed of the following, all of whom are members of the Society: Edwin Duryea, the late Virgil G. Bogue, C. H. McKinstry, John W. Reid, and the writer, who acted as Chairman.

Mr. Buel. A. W. Buel,* M. Am. Soc. C. E. (by letter).—The analytical investigations of the author promise valuable data on the destructive force of earthquakes, if he can supply more detail to support his assumptions and conclusions. His method appears to be rational, but the writer, who spent the past winter in Guatemala and had his attention called to one of the bent or twisted trolley poles and many other failures, does not find the author's assumptions entirely convincing. It is important to know the exact angle made by the plane of the bend with the web of the I-beam, and if exact sketches, with dimensions, of as many of these bent poles as can be secured, are added to his paper, the author will greatly enhance its value and make of it a really useful contribution.

While in Guatemala the writer noted that many of these I-beam trolley poles had been seriously corroded at the point where they enter the pavement (or ground), in some cases showing a loss of about one-half the original section on probably two-thirds to three-quarters of the resisting moment. It is important to know whether the bent poles had suffered loss of section and at what points. The condition of the fixed ends should also be recorded, that is, whether they were in concrete or other pavement, or in earth. Were the span wires in place when the poles were bent?

The author's example of the Las Vacas Viaduct leaves the assumptions far from clear, although this structure was designed by the writer. A sketch of the "sole-plate" referred to, showing its relation to the structure, should be added. Were the sole-plates sheared under both trusses on the bent or only under one, and was it a fixed or expansion end? Full details are necessary in order to check this analysis.

As to the weight involved, it is not even approximately true that a tower of "average" height represents average weight per linear foot, especially as the maximum height is 230 ft. and the minimum about 50 ft. from base of rail to masonry in the Las Vacas Viaduct. Moreover, if the writer understands the author to refer to a shearing force at the point where the spans rest on the top of the bent, only a fraction of the weight on the pedestals of that bent should be considered, or the weight above the point of shear. Then, on the header of a bent there is the weight of one-half of one intermediate span and one-half of one tower span. This viaduct has been described in four articles in *Engineering News-Record*,† which will perhaps be of some assistance in studying this question. After some consideration and reference to these articles, the writer is much in doubt as to whether the action

* New York City.

† Vol. 54, p. 638; Vol. 55, pp. 104 and 118; and Vol. 56, p. 297.

was simple shear or how the forces acted; only full details of the failure can establish this point. Mr. Buel.

After all, the question remains how do earthquakes differ in type and velocity of vibration? S. F. Shaw, M. Am. Soc. C. E., who was in Guatemala during the entire month, describes the four principal "quakes" as differing in type, but states that all of them were due to explosive volcanic action as distinguished from earth slips. He also thinks that he located, approximately, the center of the disturbance at a point a few miles southwest of the city. Now Las Vacas Viaduct is farther away—several miles—in the opposite direction. What effect, if any, has distance from the disturbance on acceleration and velocity?

S. Vilar y Boy,* Assoc. M. Am. Soc. C. E. (by letter).—The writer agrees with Mr. Manson that a great deal of trouble in the case of earthquake disasters is due to poor materials and workmanship, together with faulty design. Mr. Vilar y Boy.

The conclusions reached by the Committee on the San Francisco Earthquake are the result of a most complete compilation of data, far beyond anything that the writer was able to make, without giving up his everyday work. The Committee's report gives facts as to whether the different structures did or did not stand, with general remarks about them, without going into the question of how much these structures were strained above or below the yield point, whereas this is the point which the writer tried to investigate, within reasonable limits of accuracy, with reference to the Guatemala earthquakes. It appears, therefore, that the writer's point of view does not contradict the Committee's conclusions and should only be considered as a complement to its general remarks.

The writer did not think of preparing a paper on this subject for the Society at the time of the Guatemala earthquake, and, therefore, failed to take photographs or to obtain any other evidence of the failure of the trolley poles, outside of the notes required for his own guidance.

According to Fig. 1, however, the specimens taken into consideration for analysis were bent practically in the plane of the web at a point 5 ft. above the ground, and their full cross-section was ascertained by measuring with calipers in any given case. Also, these poles stood by themselves at the street corners and there were no signs of any pieces of wall having fallen on them, thereby causing them to bend under the ramming effect of collapsing walls rather than their own inertia.

They were five or six poles which happened to be included in a lot bought by a contractor, Mr. W. T. Penney, for use as floor-beams in rebuilding a private residence under the writer's supervision. That

* Guatemala, Guatemala.

Mr. Vilar y Boy.

work was finished in 1918, so that the specimens which Mr. Buel mentions are not the ones referred to in the paper, but some others which the writer disregarded altogether, due to their defective condition, or evidence of walls having struck them. Two or three of the poles taken into consideration were slightly twisted, which shows that the initial bending did not take place on the plane of the web, as assumed, but somewhat sidewise. If so, and since the plane of the web corresponds to the maximum moment of inertia, the assumption is on the right side.

The deflection at the top of the poles averaged 12 in. Their ends were in earth well packed with boulders, but no concrete, and they also had, for insulation, a wooden casing of 1-in. boards all around up to the point where they bent, the wooden casing entering the ground as well and being fastened with iron hoops.

The total top deflection does not seem to be of much value in estimating the force of the earthquakes, because after the initial bend took place, straining the poles beyond the yield point, their resisting ability decreased at every subsequent shake, of which there were perhaps fifty during the night of December 25th, 1917.

Some of the span wires were in place, running parallel to the streets, whereas the plane of the beam webs was at right angles to the wires, which therefore did not act as guy lines. There were three wires, and their size varied from, say, No. 2 to No. 6 gauge, which is rather light weight.

Relative to the Las Vacas Viaduct, it was taken into consideration simply as a means of checking the fundamental assumptions, and, as plainly stated in the paper, the writer did not even attempt to analyze the complex succession of oscillations, trepidations, and swirls.

As to the weight involved in measuring the equivalent static effect of the earthquake, it is beyond dispute that the assumption of average weight for average height is far from true, but it should be borne in mind that the earthquake impact depends on two factors, as stated in the paper, namely, the weight and the likelihood of vibration. This was a most difficult question for such a complex structure as the viaduct, and an effort was made to simplify it by "reasoning with figures" without attempting to present it as an analysis.

For this purpose, the whole of a tower was considered as a vertical pole, somewhat supported at the top, and its likelihood to vibrate was worked out on the same line as that which Professor Merriman in his "Mechanics of Materials" refers to as a cantilever with constraint, with the only difference that Professor Merriman deals with a concentrated load at the top, whereas in the case of the earthquake it is a distributed load, the weight of the tower itself.

Mr. Vilar y Boy.

Under these circumstances, the integrations of the elastic curve are as follows:

$$E\,I\,\frac{d^2\,y}{d\,x^2} = M_1 - \frac{w\,x^2}{2} \qquad E\,I\,\frac{d\,y}{d\,x} = M_1\,x - \frac{w\,x^3}{6} \qquad M_1\,l = \frac{w\,l^3}{6}$$

$$M_1 = \frac{w\,l^2}{6}$$

$$E\,I\,\frac{d\,y}{d\,x} = \frac{w\,l^2\,x}{6} - \frac{w\,x^3}{6} \qquad y = \frac{1}{E\,I}\left(\frac{w\,l^2\,x^2}{12} - \frac{w\,x^2}{24}\right)$$

$$y_{max.} = \frac{w\,l^4}{24\,E\,I} = \frac{W\,l^3}{24\,E\,I}$$

Therefore, the likelihood of the viaduct towers to vibrate as cantilevers with constraint, as compared with a plain cantilever, bears the ratio of $\frac{8}{24}$ or $\frac{1}{3}$ and if the earthquake impact for a cantilever is 9.1 times its weight, the same impact for the viaduct towers should be 3.03, or about three times its weight.

The weight to be considered, however, is only a fraction of the weight of the tower which, on account of the anchorage, was considered as fixed at the bottom and simply supported at the top.

In order to consider the extreme conditions, the corresponding distribution of weight (equivalent to the transverse strain) was calculated for the highest tower, the posts of which are built as follows, starting at the bottom:

One section, 35 ft. high.....	2	I-beams,	80	lb.	per ft.
One section, 29.5 ft. high.....	2	"	75	"	" "
One section, 29.5 ft. high.....	2	"	65	"	" "
Three sections, 29.5 ft. high....	2	"	55	"	" "
One section, 32.3 ft. high.....	2	"	55	"	" "

The center of gravity of these sections should be 116.8 ft. from the top, and the corresponding fraction of weight as a measure of earthquake impact transmitted to the top should be 0.27 of the total weight, according to the well-known formula: $2 - 3\,k + k^3$. The likelihood of vibration as previously described, however, multiplies the factor 0.27 by 3, making the fraction of tower impact at the upper end equal 0.81 times the weight of the complete tower in the extreme case. This coefficient does not greatly differ from 0.95, as found for a supported beam, and justifies for practical purposes the writer's rough assumption that the earthquake impact for the whole weight of the viaduct, disregarding the different effect of towers and trusses, especially bearing in mind that vibrations should be transmitted pretty well through an elastic material like steel, and that any increase in the reaction of the tower

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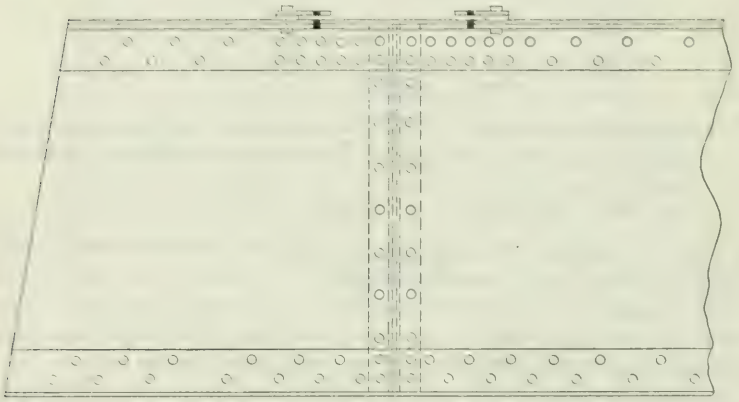
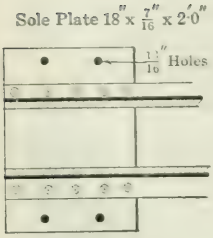
top, due to the bottom not behaving as a perfectly fixed end, would bring the factor, 0.81, up to almost 0.95.

Fig. 2, traced from detail drawings, will clear up the question of how the forces acted to cripple the rivets. The reasoning applies to a fixed end; but, in fact, there were sole-plate rivets cut both at one fixed and one expansion end, which otherwise do not greatly differ in

DETAILS OF LAS VACAS VIADUCT.



Cross-beam, showing anchorage of trusses.



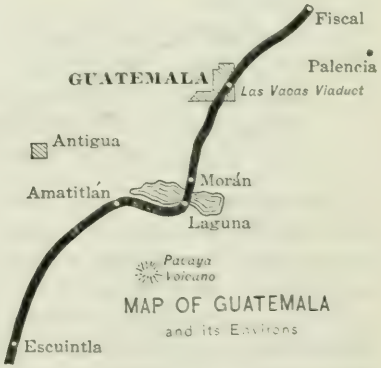


FIG. 2.

design, the only difference consisting in substituting slots for round holes. The fact that the results of these calculations agree with the actual ultimate strength of rivets confirms the computations.

Regarding velocity of vibrations and influence of distance, the writer regrets the lack of the necessary seismographic stations, so that

no definite information can be supplied. As far as he could rely on his senses at the time of the disaster, the seismic waves came from about the south, which agrees with the location of the Pacaya Volcano (Fig. 2), Lake Amatitlan, and the Town of Moran, destroyed by an earthquake in November, 1917, about a month prior to the ruin of Guatemala City. In all probability, however, the center of disturbance was between Lake Amatitlan and Guatemala City, and the subsequent earth displacement followed a fault line, as evidenced by a deep crack toward a volcanic hill near Palencia. Otherwise, Escuintla and Antigua would have suffered as much as Guatemala City, which is not the case. Mr. Vilar y Boy.

This might compensate for the increased distance from the supposed center of disturbance to Las Vacas Viaduct, 3 miles from Guatemala, along the railroad, and explain the damage suffered by the viaduct. Lake Amatitlan showed evidence of volcanic activities at its south shore, where there was a strong steam-blow through a crack running toward the Pacaya Volcano; but, in all probability, it was simply a case of a geyser originating at the bottom of the lake, 25 ft. deep at that point, and the surface crack was only the outlet of a zigzag conduit coming up from below. The steam jet was intermittent, increasing and decreasing by 8-min. cycles, as though the pressure of steam generated by water percolating into some deposit of anhydrous rocks stopped any further action until that pressure was released through the surface outlet.

The temperature was 134° Cent. for steam jet, with no signs of hydrochloric or sulphurous acids, 62° Cent. for the soil, 50° Cent. for the water on the lake surface, and 28° Cent. for the atmosphere, from which the average heat transmitted per square meter per hour should be about 218 calories and, therefore, the thickness of material between the atmosphere and the steam should be about 1.37 m., which seems to prove that the outlet conduit did not come straight from the bottom, but through a crooked zigzag line, and, therefore, its direction at the surface should be disregarded. The computed thickness was confirmed when that place caved in later.

The report of the Committee on the San Francisco Earthquake should be considered first to determine the general stability conditions of any earthquake-proof construction. After these general conditions are established, the writer's method of calculation will assist in designing properly the required structure, proper design being one of the conditions emphasized by the Committee.

The likelihood of vibration as a factor of the earthquake impact agrees fairly well with failures observed in Guatemala, and in the case of the Las Vacas Viaduct the discrepancy between the two factors, 0.81 and 0.95, can be explained by the bottom of the towers not behaving

Mr. Vilar y Boy.

altogether as perfectly fixed ends. This is not a mathematical reason, but such is the case with the assumption that a pillar with flat ends represents the intermediate condition between supported and fixed ends, which assumption does not prevent its being accepted in ordinary practice.

Moreover, with reference to earthquakes it would be useless to look for analytical refinements, since the forces of Nature cannot be standardized, and the results obtained from a complete analysis of one earthquake would not agree mathematically with the next one. For the sake of protection of life and property, however, it appears that the customary permissible stresses applied to the strains resulting from the study of the Guatemala earthquakes in 1917-18, which completely destroyed a town of one-story buildings, should afford a reliable guide as far as human foresight can control natural phenomena.

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Paper No. 1458.

HYDRAULIC-FILL DAMS*

By Allen Hazen, M. Am. Soc. C. E.

With Discussion by Messrs. C. E. Curtis, Richard D. Chase, G. A. M. Elliott, J. M. Howells, D. C. Henny, A. T. Goldbeck, George L. Dillman, H. B. Muckleston, H. F. Dunham, William H. Burr, M. M. O'Shaughnessy, Arthur E. Morgan, H. deB. Parsons, George Sidney Binckley, Charles H. Paul, John E. Field, J. Albert Holmes, H. W. Swanitz, Joel D. Justin, and Allen Hazen.

Synopsis.

The hydraulic method of dam construction has great advantages, but too many dams built by it have failed.

As often built, the center of the dam is composed of a clay core completely water-tight, but very wet as a result of the method of construction, and so completely water-tight that it is incapable of permitting its own drainage.

This soft core in the center of the dam exerts the pressure of a liquid of high gravity on the toes and tends to push them out. This pressure seems to have been responsible for most of the failures of hydraulic-fill dams.

The method of placing the core commonly used results in an exceedingly fine-grained material, desirable from the standpoint of a water-tight dam; but it may be that extreme tightness is secured at the expense of stability.

To use the hydraulic method of dam construction successfully, it is necessary either to increase the toes until they are large and heavy

* Presented at the meeting of May 5th, 1920.

enough successfully to resist the full fluid pressure of the core, or else to handle the core material so that the finest particles are wasted, leaving only the larger ones, and in that way increasing the grain size to a point where drainage can be secured, while on the other hand remaining sufficiently impervious for practical purposes.

Methods of testing are described and an effort is made to analyze the conditions of stability.

The hydraulic-fill method of dam construction grew out of hydraulic mining. It was natural that a process conspicuously successful in mining on an enormous scale, and which moved material at very low unit costs, should find other applications.

The late James D. Schuyler and J. M. Howells, Members, Am. Soc. C. E., were pioneers in the application of hydraulic methods to dam construction. Mr. Schuyler presented a paper* to the Society in 1906 setting forth in a most admirable way the peculiarities of the method and its application to actual dams.

The method is certain to have an important place in future dam construction. It has two fundamental advantages: First, in sorting out the fine particles and placing them in the center of the dam, thus insuring complete water-tightness; and, second, in the use of power in place of muscle to the greatest extent, so that the labor cost and the animal cost of construction are reduced to a minimum.

It is sometimes possible to place material by the hydraulic process at much smaller cost than by any other method.

On the other hand, several hydraulic dams have not proved to be stable. Failures, usually during construction, have caused great losses, and their occurrence has been discouraging.

There may have been more than one cause of failure, but one cause has certainly predominated. In the interior of the dams there have been masses of clay and other fine-grained material, that have not consolidated to the point of stability. Instead of forming an integral part of a solid dam, these masses have remained in almost liquid form, dividing the dams and tending to disrupt them. In this respect, hydraulic-fill dams are on a different basis from other earth dams.

If the hydraulic method of dam construction is to be used successfully, this element of weakness must be eliminated.

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* *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 196.

There seem to be two promising ways of eliminating it. The first and most natural one is to increase the size of the dam until the solid parts forming the toes are amply strong to resist the full fluid pressure of the unstable core. The second consists in selecting material and handling it in a way to secure drainage and consolidation of the core, so that the whole dam will act as one solid mass.

If core material can be fully drained and consolidated, there is no reason why a dam built by the hydraulic method on a section that is suitable for an earth dam built dry should not be safe. If drainage and consolidation of the core are not secured, however, safety can only be reached by making each toe large enough to stand the full pressure of the liquid core, and in that case, in order to be safe, a dam built by the hydraulic method must have flatter slopes and a larger section than is otherwise needed for an earth dam.

It is interesting to note that Clemens Herschel, Past-President, Am. Soc. C. E., in discussing Mr. Schuyler's paper in 1906, made the point that a larger section would be required for stability; Mr. Schuyler took exception to this, and stated that in his experience there was no reason for building a dam of greater dimensions because the material was placed in it by the hydraulic method.

The advantages and economies of the hydraulic method are such that it will often be good business to use it, even though the required volume of the dam should be greater. If work can be built at half the cost per cubic yard, twice as many cubic yards can be placed for the same money.

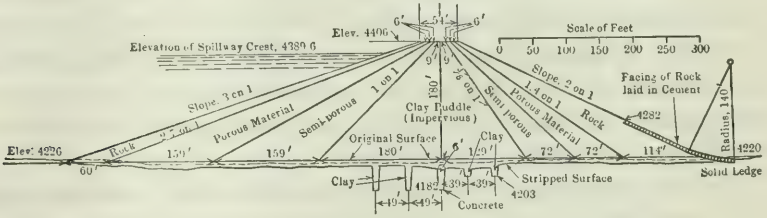
The number of failures with hydraulic-fill dams growing out of this fundamental condition of fluid cores and inadequate toes has been such as to make it clear that this condition of instability has not been recognized in the designs. Mr. Schuyler's excellent paper contains no discussion of this point. Looking at it from the standpoint of thirteen years afterward, it is easy to see how this condition could have been overlooked and to see, also, how some of the early failures were attributed to other causes.

Now, however, enough experience has been gained so that a better analysis may be made. To insure success with future hydraulic dams, this analysis must be thorough. It is the writer's thought that it will be possible to make it in a way to secure as high a degree of safety in hydraulic-fill dams as is required, and is customary, in other engineer-

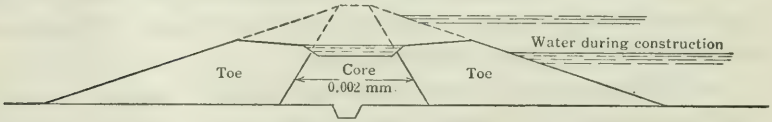
ing structures. It may be that we do not yet have all the necessary data; but, if not, the best way to find what is lacking is by using what we have.

Character of Core Material.

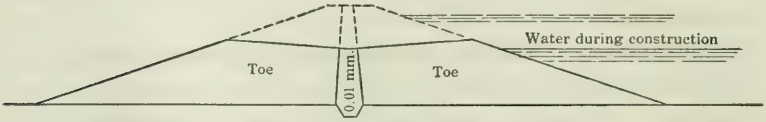
The first point to be considered is that, in many or perhaps most hydraulic-fill dams, the core material is so fine in grain size that it is incapable of drainage. By that is meant that it is incapable of drainage within a reasonable length of time. The lower parts of the core do not become stable before the upper parts are placed upon them.



CROSS-SECTION OF DAM NO. 2 AT NECAXA, MEXICO.
SHOWING
DIMENSIONS, CUT-OFF TRENCHES AND THEORETICAL DISTRIBUTION OF MATERIALS.
FIG. 1



TYPICAL SECTION OF HYDRAULIC FILL DAM DURING CONSTRUCTION.
FIG. 2



ILLUSTRATING PROPOSED SECTION WITH NARROW COARSE-GRAINED CORE.
FIG. 3

In general, all the material placed in hydraulic-fill dams may be divided into two classes, namely, toe material and core material. Mr. Schuyler presented a diagram of materials divided into several classes, the coarsest being found at the outside and the finest in the middle (Fig. 1*). Practical experience does not show that the material is graded in quite that way. There is some grading in the toes and the average size of particles near points of delivery of material near the outer slopes is

* Fig. 1 is reproduced from *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 243.

greater than away from those points and near the core; but, practically speaking, this gradation is not very important. Ordinarily, all the toe material is of coarse particles when first placed. Afterward, passage of silt-laden water from the operation of hydraulicking partly fills the voids with fine material, but the process stops itself before they are all filled. The toes drain readily and all the stability naturally inherent in the stock is obtained.

The core may be defined as that part of the dam deposited from water in the central pool. In the writer's experience, core material is homogeneous. At any level there is no appreciable difference in grain size between the middle of the core and its farthest extremities.

The principles determining the size of particles deposited in the core, or not deposited, are the same as those that apply in a sedimentation basin of a water-works plant. The larger the area of the pool in proportion to the quantity of water that goes through it, the more complete will be the subsidence, and the smaller will be the limit of size of particles retained.*

Most hydraulic-fill dams have been built with good-sized pools in the center during construction (Fig. 2), in which sedimentation has been fairly complete. The writer has examined samples of the core materials from a number of dams. The samples are rather surprisingly alike in grain size. They ordinarily contain particles in large quantities down to a limit between 0.001 and 0.002 mm. (or sometimes 0.003 mm.) in diameter. Generally speaking, all smaller particles have been carried away by the escaping water and have not remained in the core.

It should be stated that the determination of the exact size of these small particles is by no means as precise as the determination of the size of grains of filter sand. The results are reached by the microscope with a micrometer and are to be considered as roughly approximate only. It has been the intention to use a basis as nearly as possible comparable to that adopted for the larger particles in sand analysis, which is to use as the diameter of any particle the diameter of a sphere of equal volume. Following this rule the width, rather than the length, of small particles is measured.

Core material then, as a matter of observation, consists of particles from this size up to the size of the smallest particles retained in the

* "On Sedimentation," by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 45.

toes. In a general way, this size may be taken as 0.03 mm., or ten to fifteen times as great as the limit of size of particles that are retained in the core.

Speaking of core materials in the terms that are used in describing sand, it may be stated that the effective size of core material is ordinarily about 0.002 mm. This size is so small that one who is not accustomed to microscopic work has little conception of it. For comparison, it may be stated that ordinary filter sand has an effective size of from 0.3 to 0.4 mm., while dune sand, being the finest material usually spoken of as sand, has effective sizes ranging from 0.15 to 0.20 mm.

Without attempting precision, and using round figures, the core material is one-hundredth of the size of dune sand. This means (assuming that the laws of flow and the laws of capillarity applying to the finest sands apply also to these still finer materials) that, other things being equal, the drainage of core material will take ten thousand times as long as corresponding drainage of fine sand. Fine dune sand would drain as much in an hour as core material in a year. It means, further, that the height to which water can be held by capillarity is ten thousand times as great as the height at which it is held by fine sand. If fine sand will hold water by capillarity to a height of an inch, as it will, then core material like that shown by the samples will hold water by capillarity to much more than the full height of any dam.

The reason why this core material does not drain is therefore clear. It seals itself up and becomes practically water-tight.

With the dimensions existing in dams that have been built, assuming that the laws of flow known to apply to the finest sands apply also to the still finer materials, it will take years for the excess of moisture to drain out, as it must do before the solidification of the material is possible.

As a practical proposition, drainage is not possible with such fine materials. By that is meant such prompt and complete drainage as will result in solidification during the construction of the dam.

Construction records and samples show approximately what happens. Core material of the fineness already described goes down first in the form of soft mud in which the voids filled with water are at least 70% of the total volume. Such mud is very soft.

The percentage of voids furnishes, on the whole, the best index of consolidation and stability. In itself, it is not an adequate basis of

comparison, because different kinds of core material may have different degrees of stiffness with the same percentage of voids; but, notwithstanding this difference, the percentage of voids is the best index of consolidation so far available.

The material when it goes down contains 70% of voids. Under these conditions, the minute solid particles are held apart by the water contained between them, and the passages through which water can pass are much larger than they would be with a more compact arrangement of the same material. Water drains from this material gradually, and the material consolidates.

The drainage may be horizontal, in which case the surplus water finds its way through the core material to the toes and escapes. There is, however, a shorter way out, and it appears probable that most of the drainage takes place vertically. That is to say, the solid particles settle down and consolidate while the water moves upward between them to the top. This consolidation takes place gradually at rates depending on the dimensions of the dam and also on variations in the effective size of the core material; for while the writer considers all this material broadly as having an effective size of 0.002 mm., there are no doubt considerable variations in size and permeability.

Experience shows that consolidation goes on so that after a period of from a few months to two or three years in the ordinary course of dam construction, the material will have been consolidated to a point where the voids are about 50 per cent. As the consolidation increases, the sizes of the passages between the grains become smaller, the flow becomes less rapid, and the process of consolidation goes forward more and more slowly.

Material with 50% of voids when taken out of a boring is described as material having the consistency of stiff putty. It is capable of standing up in a boring driven beyond the casing for several feet, and it offers great resistance to penetration.

One of the best field methods of testing the consolidation of core material has been to find the depth to which a 1½-in. pipe could be forced into it by two men. This method of testing has been used in several dams. It has been possible to co-ordinate the penetration so found with records of the percentage of voids. In a general way, penetration by 1½-in. pipe pushed down by two strong men extends to the point where the voids are about 50 per cent. In other words, the 50%

material is so stiff that it can no longer be penetrated to any considerable depth by this method.

Tests of material by letting down cannon balls, etc., are less searching. The penetration does not go as far.

Material with 50% of voids in a considerable depth acts essentially as a liquid. It exerts the full lateral pressure corresponding to its height and weight per cubic foot, and when the resistance of the toe is overcome, this material moves forward and flows. Precise limits cannot be set, but this is the general result of observation at Calaveras as a result of determinations of voids in many samples of material that did flow and in other samples of material that did not flow. This refers to the Calaveras Dam* near San Francisco, Cal., which slipped as it was approaching completion on March 24th, 1918, investigation of which afterward gave an unusual opportunity for studying some of the conditions that grew out of the methods of placing material that had been used. (See Figs. 4 to 6.)

Core material becomes solid when it is dried. Core material dried in the summer sun at Calaveras was consolidated until only 35% of voids remained. In this condition, it was only a little less strong and stable than the natural sandstone in the neighboring hills.

Core material consolidated to 35% of voids would form the strongest and most stable kind of an earth dam. No large toes would be required to contain it, for on any reasonable slope it would be stable by itself. Such consolidation, however, as far as is known, is reached only by complete drying. Unfortunately, drying cannot be applied to the core of a large dam.

Without attempting precision of statement, it is the writer's judgment that if core material like that at Calaveras could be consolidated to 40% of voids, it would be sufficiently stable to do its share toward resisting the pressures that come on the dam without tending to disruption. Perhaps this degree of stability would be reached before consolidation had proceeded as far as 40% of voids. There is a middle ground of uncertainty. Material with 50% of voids is still unstable; and between this limit and 40%, one cannot be certain just where to draw the line.

* See *Engineering News-Record*, April 4th, 1918, p. 679, and December 26th, 1918, p. 1158.

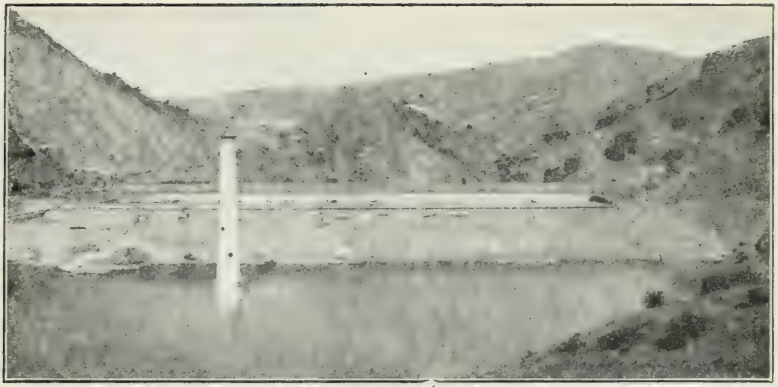


FIG. 4.—CALAVERAS DAM DURING CONSTRUCTION, SHOWING UPPER AND LOWER TOES, WITH POOL BETWEEN. WATER IN FOREGROUND IS IN RESERVOIR ABOVE DAM.

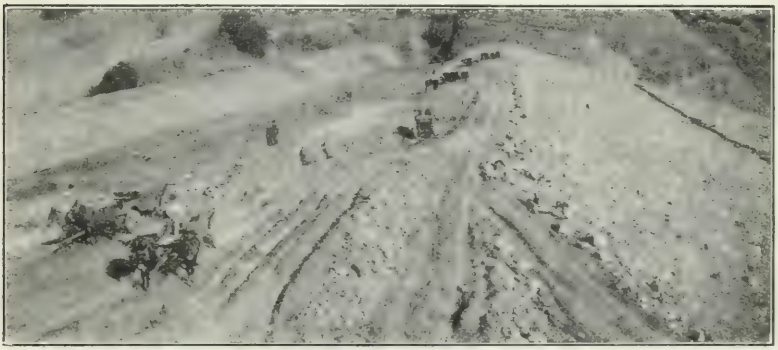


FIG. 5.—CALAVERAS DAM DURING CONSTRUCTION, SHOWING TOES AND CENTRAL POOL A FEW DAYS BEFORE THE SLIP.

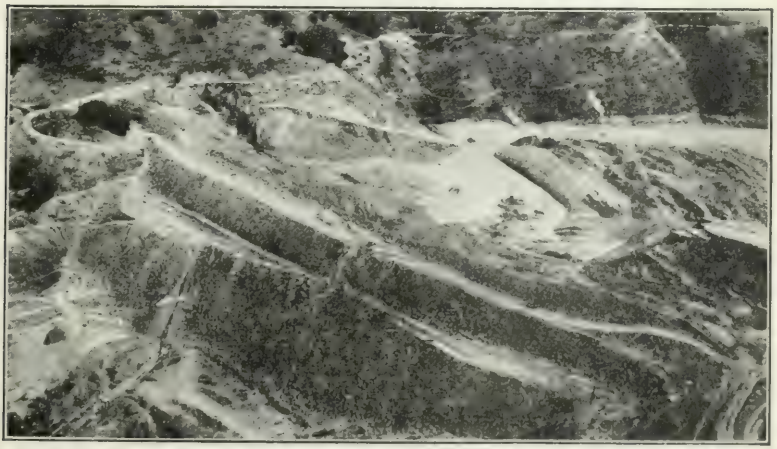


FIG. 6.—CALAVERAS DAM AFTER THE SLIP. ON MARCH 24TH, 1918, 800 000 CU. YD. OF MATERIAL MOVED 300 FT. UP STREAM AND DROPPED 100 FT. IN ELEVATION, TURNING ALSO 30° TO THE RIGHT.

It is interesting to note that the 1½-in. pipe used in the penetration tests only reaches to points where the material is still unstable. It follows that tests of this kind do not and cannot demonstrate stable material. Tests made by cannon balls or other less searching means are equally defective. It cannot be considered, in the light of the Calaveras experience, that tests made by such methods throw much light on conditions of stability. They may at times serve a purpose in showing lack of stability, but that is as far as use can be made of them.

It is the writer's thought that the best method of investigating consolidation and stability is by making borings. There are difficulties, for boring usually involves stopping the sluicing of material while the holes are being put down; but it does lead to more definite results. Borings can be driven and samples taken from the bottom of the borings for physical inspection and for determination of the percentage of voids.

The Determination of the Percentage of Voids.

This is a very simple matter, but experience shows that it is easy to go wrong, and for that reason a brief statement of methods that have been found suitable will be given.

The samples taken from the bottom of test borings are saturated with water. A determination of water content, therefore, can be made a measure of voids. This procedure is sound so far and only so far as the voids are completely filled with water. A useful check can be obtained from the specific gravity of the moist material.

The specific gravity of the solid particles of core material may be taken as 2.65. It may be that in some locations other specific gravities will be found, but, in the writer's experience, actual values differ little from this mean value.

In determining the specific gravity of these exceedingly small particles, the most careful manipulation is required to get all the air out of the dry material. Methods similar to those used for determining the specific gravity of Portland cement are appropriate, but water can be used.

A table can be made showing the weight of the solid particles, the weight of water, the percentage of water by weight, and the specific gravity of core material containing various percentages of voids.

Table 1, in convenient and somewhat condensed form, gives these values for saturated clay.

TABLE 1.—SATURATED CLAY.*

Specific Gravity of Solid Particles, 2.65.

Percentage of voids.	Percentage of water, by weight.	Specific gravity. Wet.	WEIGHT PER CUBIC FOOT.	
			Wet.	Dry.
40	20.1	1.990	124.2	99.2
41	20.8	1.974	123.1	97.5
42	21.5	1.957	122.1	95.9
43	22.2	1.940	121.1	94.2
44	22.9	1.924	120.0	92.6
45	23.6	1.908	119.0	91.0
46	24.3	1.891	118.0	89.3
47	25.0	1.874	117.0	87.6
48	25.8	1.858	116.0	86.0
49	26.6	1.841	115.0	84.3
50	27.4	1.825	114.0	82.7
51	28.2	1.808	112.9	81.0
52	29.0	1.791	111.9	79.3
53	29.8	1.775	110.8	77.7
54	30.6	1.759	109.8	76.0
55	31.5	1.743	108.7	74.4
56	32.4	1.726	107.6	72.8
57	33.3	1.709	106.6	71.1
58	34.2	1.692	105.6	69.5
59	35.1	1.676	104.5	67.8
60	36.1	1.660	103.5	66.1
61	37.1	1.643	102.5	64.5
62	38.1	1.626	101.5	62.8
63	39.1	1.609	100.5	61.2
64	40.1	1.593	99.5	59.5
65	41.2	1.577	98.5	57.8
66	42.3	1.560	97.4	56.2
67	43.4	1.543	96.4	53.5
68	44.5	1.527	95.4	52.9
69	45.6	1.511	94.4	51.2
70	46.8	1.495	93.4	49.6

* This table is not to be used for any material that has lost water by evaporation so that the voids are not completely filled with water.

When a sample is taken from a test boring, a portion is put in a dish and weighed. The weighed material is dried at or above the boiling point to constant weight, and the percentage of moisture computed. As even a little drying before the first weight affects the result, the samples should be weighed on the spot when first brought to the surface; to do this, portable equipment must be used. After the first weighing, samples may be taken to the office or laboratory for the completion of the process. When the percentage of moisture

by weight is found, the corresponding percentage of voids is obtained from Table 1.

As a check, the specific gravity of another part of the sample is determined. This is obtained by putting a weighed portion of the moist mass into a graduated cylinder partly filled with water and noting the increase in water level. The specific gravity is the ratio between the weight of material added and the weight of the water that corresponds to the increase in level, and so to the volume of material placed in the cylinder. With this specific gravity determined, Table 1 indicates the percentage of voids.

The percentage of voids determined by the two methods with good manipulation should check within 1 or 2 per cent. If it does not, there is something wrong and the error must be found and eliminated.

To determine the weight of the coarse and hard toe material, no better method has been found than to make an excavation accurately to a dimension of 1 cu. yd., or other convenient size, and to weigh the excavated material on platform scales. Percentages of moisture vary, and the weight of dry material affords the best basis of comparison. An average sample of material on the scales is taken for determining the percentage of moisture, and the results are reduced to weight of dry material per cubic foot.

Draining Fine Material.

The writer is fully satisfied that there is no practical way of rapidly draining core material as fine in grain size as that described previously. He has considered it from a theoretical standpoint and also from the standpoint of practical experience.

After the material reaches a condition of 50% of voids, 6 cu. yd. must be compressed into 5 cu. yd. of 40% material by the exclusion of 1 cu. yd. of water before full stability can be counted on. This water, amounting to one-sixth of the whole volume of core material, must be expelled. The computed rate of flow through material with an effective size of 0.002 mm. is very slow. The rate depends on, and is proportional to, the hydraulic slope. With a slope of 10% (which is as steep as is usually found) the computed rate of flow as a solid column of the full area of the section is 0.32 mm. daily. At this rate, 80 days are required for the water to move 1 in. and 2.6 years for it to move 1 ft. This calculation cannot be used with assurance as estab-

lishing the exact rate of flow, but it is the writer's judgment that the actual rate is as likely to be below as above the result given. It is clear that, for a large dam, complete drainage and consolidation may be a matter of many years.

From a practical standpoint, it may be recorded that a 6-in. well driven 60 ft. in saturated core material having voids averaging 50%, with the casing perforated freely to admit water, stood empty for weeks. Not enough water drained from this material to raise the water in the well. This may represent the effect of capillarity. With such a degree of tightness, it is clearly useless to attempt to secure consolidation by any method depending on additional drainage outlets.

Measuring Core Pressures.

An interesting method of measuring the pressures actually existing in core materials has been suggested by A. T. Goldbeck, Assoc. M. Am. Soc. C. E.* By this method, small test cells are built into the core material. Some are placed with horizontal and some with vertical faces, arranged to indicate pressures on the faces during construction and afterward. Such devices are reported to have been placed in hydraulic-fill dams now building by the Miami Conservancy District. The results of observation through a certain period have already been recorded. These observations indicate, as would be expected, that the horizontal and vertical pressures are the same at first and until the material has become somewhat consolidated. Afterward, horizontal pressure increases less rapidly than vertical pressure, and the inference may be drawn that this represents solidification of the material in which the cells are placed to a point where the full horizontal pressure is no longer produced.

This is certainly an interesting method of test and one that may throw light on the conditions of core material.

It reminds the writer of one of his early experiences, which related to different circumstances, but possibly the same underlying principles are involved. It was proposed to build large and heavy structures in which the weight would be well distributed on a foundation of stiff silt. The question was presented as to whether the foundation was sufficient to carry the weight. An apparatus was provided by which test areas of 1 sq. ft. at the level of the proposed foundation were

* *Engineering News-Record*, April 18th, 1918, p. 758.

loaded. The results of the tests were most satisfactory. The material carried more than the expected loads, with only insignificant settlements. When the actual structure was built, however, there was considerable settlement. The actual settlement was many times greater than had been indicated by the tests.

The explanation is simple enough. The single square foot that was loaded distributed the weight applied to it to a much larger area a short distance below. What was loaded was in effect a pyramid with a flat top of 1 sq. ft. The full applied weight per square foot was carried only at the top, and the unit stress rapidly became less going downward as the area of the pyramid increased. A thin layer of material just below the footing actually got the pressure and was compressed accordingly, and in this way the slight observed settlement was produced. Lower down, however, the unit pressure was lower and there was no appreciable compression.

When the structure was built there was no chance for a corresponding distribution of pressure. The whole area underneath was loaded, and the weight had to be carried through the full depth of silt and the silt was compressed. A corresponding settlement would no doubt have taken place in the test apparatus if the material loaded had been a column 1 ft. square all the way down to rock. The writer thinks that because of this dispersion of load it is generally true that a small test area will carry more per unit than a larger one, and it may be that the same conditions will be found to apply to the test cells built into the core of a dam.

In view of all the information now available, the only safe course to follow, so long as the core material is like that here described, is to assume that it will produce the full horizontal pressure corresponding to its weight and height; and the toes must be made large enough to resist that pressure with ample safety.

Resisting the Pressure of the Core Material.

The first point to be considered is the coefficient of friction of the material in the toe, sliding either upon the foundation or upon itself. With hard, clean gravel a coefficient of 0.7 or 0.8 might reasonably be expected, although the data for establishing this coefficient are not as convincing as could be desired. No such coefficient was found, however, in accounting for the Calaveras slip. Instead, it was esti-

mated that the weight of the material pushed forward in the up-stream toe was five times as great as the pressure of core material against it. In estimating that pressure the whole height and weight of core material was included, assuming that it acted as a heavy liquid. This indicated a coefficient of friction of only 0.20. This is to be taken as an average for the whole area on which the slip occurred. It may be recorded, however, that a similar calculation for the lower toe which did not slip indicated a coefficient of 0.22. No one can tell how much more it would have held.

The Calaveras material is softer and perhaps more slippery than the hard particles of glacial drift of the Eastern States; and it would not be reasonable to expect as high a coefficient of friction. It may be, also, that the coefficient of friction under such heavy pressures as are found in the lower part of a high dam will be less than under the small pressures of moderate depths.

Some experiments were authorized by George Wilhelm, M. Am. Soc. C. E., and made by G. W. Hawley, M. Am. Soc. C. E., Resident Engineer at the San Pablo Dam, near Berkeley, Cal. (see Fig. 7), following suggestions of the writer. A 14-in., cast-iron pipe was cut into two short parts, one of which was fixed and the other attached securely to a steel frame which swung freely on an axis about 8 ft. above. Another steel frame securely connected the ends of this axis with the fixed part of the 14-in. pipe and with other parts of the apparatus. Two hydraulic jacks were used and the pressures were computed from the measured pressures on the pistons.

The two pieces of 14-in. pipe were placed in line and the frames held them so that the ends would just clear.

A sample of the material to be tested was placed in the pipe, filling the lower or fixed part and extending some distance into the movable part. Oak planks, cut to fit as a loose piston, were then placed above. One of the hydraulic jacks was placed above it. Pressure was then applied to compress the material in the pipe to any desired extent. Experiments were made with various pressures, the greatest corresponding to a depth of fill of 200 ft. While the material was held under this pressure, the other jack was placed horizontally against the side of the 14-in. pipe with suitable blocking, and pressure applied until there was movement. The ratio of pressures on the two jacks then gave the coefficient of friction.

FIG. 7.—SAN PABLO DAM, SHOWING DEVELOPMENT OF LOWER TOE, WITH THE FINER MATERIAL PASSING OVER IT TO THE CENTRAL POOL.

The steel frame and the pivot at the top were built so that there was no appreciable friction in the apparatus. A slight correction was necessary because the lever arm to the slipping plane was longer than the lever arm to the point on the 14-in. pipe where pressure was applied.

The form of the experimental apparatus was suggested to the writer by a paper by Mr. A. L. Bell on clay pressures.* The apparatus as worked out was more like that used by E. P. Goodrich, M. Am. Soc. C. E., in his experiments on soil pressures.†

The apparatus used differed from both of its predecessors in that tests were made at very much higher pressures. The results may be compared, but it must be remembered that the local material had peculiarities of its own, and that it was no doubt quite different physically from the material tested by Messrs. Bell and Goodrich.

The writer will not attempt to discuss this matter further, but suggests it as a fruitful topic for further experiment.

Experiments were made by Mr. Hawley with various local materials at various pressures. These showed that, with low pressures, the coefficient of friction was distinctly larger than it was with higher pressures. The explanation for this may be that the higher pressures broke down and crushed the particles, and so reduced the relative resistance. After a certain pressure was reached there was less rapid change in the coefficients, and for the higher pressures the coefficients for any material were practically constant.

For material from the local soft sandstone rocks at San Pablo, coefficients of friction of approximately 0.5 were found.

There was one kind of material more slippery than the others for which an average coefficient of 0.45 was found. Other harder materials gave coefficients of 0.55 and more. Sand and gravel from the creek representing the remnants of much harder rocks in the hills (and so corresponding more nearly to the glacial drift of the Eastern States) gave coefficients of about 0.7.

The materials at San Pablo are similar to the materials at Calaveras; and there is reason to think that the coefficients found at San Pablo would apply approximately to the Calaveras materials.

* *Minutes of Proceedings*, Inst. C. E., Vol. CXCIX (1914–15), p. 233.
† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 297.

What then is the explanation of the fact that the Calaveras Dam slip indicated a coefficient of 0.2 while similar materials in the test apparatus show a coefficient of 0.5—two and one-half times as much?

The answer seems to lie in the fact that in the course of construction at Calaveras, before the dam was very high, the construction pool in the center of the dam was sometimes quite wide; so that at some levels the greater part of the width of the dam was composed of fine-grained core material. Afterward the proportion of solid fill was increased and the width of core reduced, but the effects of the wide pool may have been permanently left in layers of core material extending far out under the more solid parts of the toes.

It seems probable that unstable core material placed in this way furnished the lubricant that facilitated and made possible the slip in the dam.

Section of Dam Required with Core Material Considered as a Fluid.

At Calaveras, the toe material was slightly heavier per cubic foot than the core material. At San Pablo, with fragments of porous sandstone rock predominating in the toes, there was scarcely any difference. With harder material like the glacial drift of the Eastern States, the toe material would be considerably heavier.

For the purpose of a first rough calculation, assume that the weights per cubic foot of toe and core material are equal, as they are at San Pablo. If a coefficient of friction of 0.5 is assumed, each toe must weigh twice as much as the pressure exerted by the core. The pressure exerted by the core is $\frac{1}{2}\ wh^2$ and the weight of the toe to balance it must be wh^2. To produce this weight, a uniform outer slope of 2 to 1 will suffice. In addition, a top width equal to the maximum width of core must be used. If a factor of safety of 2 is required, the outside slope must be made 4 to 1, but in view of the records of slips in actual dams, it may well take a higher factor of safety. A slope of 5 to 1 will give a factor of safety of 2.5 and a slope of 6 to 1 will give a factor of 3. Where toe material is considerably heavier than the core, as would be the case with harder and heavier rocks, the slopes could be reduced in proportion to the increased specific gravity without reducing the factor of safety. Thus, with toe material 20% heavier than core material, a slope of 1 to 5 would allow a factor of safety of 3. Very few hydraulic dams have been built with sections as large as this

line of thought suggests. The Gatun Dam at Panama heads the list, and is even larger.

A great many cubic yards are involved in such a large section, but if the work is done cheaply enough per cubic yard, it may be economical to use the extra volume. Safety can be secured in this way; and it may be that this is the best procedure to follow.

Dimensions of a Number of Dams.

To see what these slopes would call for in the way of section in comparison with earth dams that have been actually built either by the hydraulic method or as dry fill, the following comparison is made. Assuming a freeboard of one-tenth the greatest depth of water, H, and a top width of 0.25 H and uniform slopes, the cross-sections of dams in terms of the greatest depth of water would be as follows:

Slope.	Volume.
3 to 1	3.9 H^2
4 to 1	5.1 H^2
5 to 1	6.3 H^2
6 to 1	7.5 H^2

The actual dimensions of a number of dams are given in Table 2. Rock-fill dams are not included, but rock fill forms some part of a number of dams in the list. No dams holding less than 64 ft. of water are included.

The Necaxa Dam slipped, but was afterward rebuilt on the original section. The Big Meadows Dam is being raised with an enlarged section.

Increasing the Grain Size of Core Material.

The second way of increasing stability is to use core material that will consolidate more rapidly and thoroughly. There is only one way to insure this; that is to increase the grain size of the material allowed to remain in the core. With pools and water quantities actually used in the dams known to the writer, particles smaller than 0.001 mm. and perhaps up to 0.002 mm. have been wasted. Everything larger has gone into the dam.

In some cases, an effort has been made to secure completeness of deposition of these fine particles to avoid waste and to increase the yardage accounted for in the dam.

TABLE 2.—Dimensions of a Number of Earth Dams That Have Stood Successfully.

Dam.	Height to flow line, H.	Area of section, in square feet, A.	Ratio, $\frac{A}{H^2}$	Freeboard.	Width, in Feet.				Bottom, W.	Ratio, $\frac{W}{H}$
					Top.	At flow line.	50 ft. below flow line.	100 ft. below flow line.		
*Gatun	78	99 200	16.30	30	100	397	1 469	....	1 990	25.5
*Big Meadows	64	24 300	5.93	20	30	160	460	....	548	8.56
*Coquitlam	83	38 700	5.61	15	40	145	521	....	702	8.48
*Cambria	82	31 200	4.65	13	20	98	429	....	685	8.34
Ashokan	90	35 400	4.37	20	34	112	400	....	650	7.23
Druid Lake	82	27 900	4.15	5	60	90	390	....	582	7.10
San Andreas	80	26 200	4.10	5	30	58	387	....	606	7.58
*Paddy Creek	120	58 200	4.05	25	23	161	414	664	757	6.30
*Somerset	92	33 200	3.94	13	25	78	375	....	616	6.67
San Leandro	65	16 350	3.88	5	30	56	335	....	457	7.04
*Haiwee	75	21 600	3.85	14	20	90	340	....	465	6.20
Croton	96	33 400	3.62	24	30	126	336	....	528	5.50
Tabeaud	86	26 400	3.57	8	20	60	345	....	553	6.44
*Necaxa	164	90 800	3.36	16	54	136	386	636	954	5.81
Cold Spring	88.5	26 200	3.34	10	20	70	320	....	512	5.79
Belle Fourche	100	33 100	3.31	15	20	65	284	656	656	6.56
Lahontan	112	40 900	3.25	12	20	80	330	580	640	5.72
Santa Maria	85	23 500	3.25	8	20	60	310	....	485	5.71
Pilarcitos	74	17 600	3.22	5	30	56	288	....	415	5 61
Morris	90	25 700	3.18	9	20	60	309	....	502	5.58
Borden Brook	64	12 700	3.10	7	24	49	280	....	337	5.27
Honey Lake	90	25 000	3.08	6	20	50	300	....	500	5.56
Goose Creek	137.5	54 900	2.90	7	16	54	304	554	741	5.39

* Hydraulic fill, wholly or in large part.

To increase the effective size of core material, it is necessary to increase the limit of size of particles to be wasted. This can be done by narrowing the pool or by increasing the quantity of water passing it (see Figs. 3, 8, 9, and 10). On one occasion at San Pablo, to test this out, the toes were allowed to increase in height for a time without raising the pool. This was continued until the pool almost disappeared. Hardly more than a muddy creek remained in the middle of the dam. The core became comparatively narrow and much coarser in grain size. The effective size was increased to approximately 0.01 mm. Finer particles were all wasted. In other words, the core material was, say, five times as great in grain-size. It had, therefore, twenty-five times as great draining capacity, and only one twenty-fifth of the capacity to hold water by capillarity.

Material placed in this way had a much better chance of drainage; first, because the core was narrower and there was less water to be

FIG. 8.—SAN PABLO DAM, SHOWING MAXIMUM WIDTH OF CENTRAL POOL.

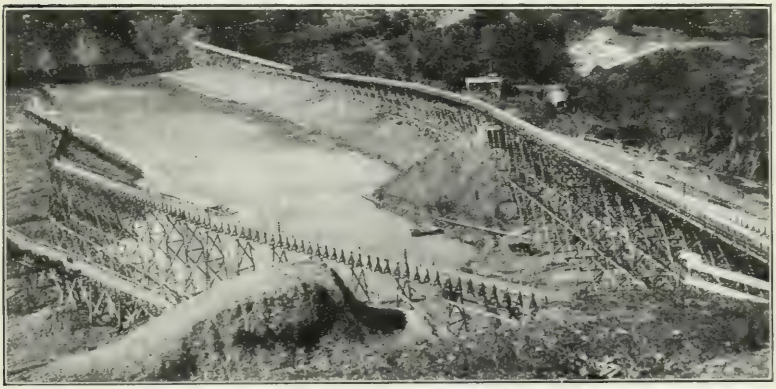


FIG. 9.—SAN PABLO DAM, SHOWING ORDINARY WIDTH OF CENTRAL POOL PRODUCING CORE MATERIAL 0.002 MM. IN SIZE.

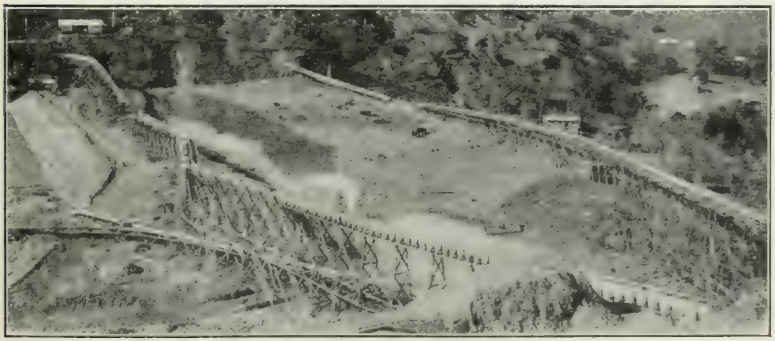


FIG. 10.—SAN PABLO DAM, SHOWING CENTRAL POOL GREATLY REDUCED IN WIDTH FOR PURPOSE OF INCREASING GRAIN SIZE OF CORE MATERIAL. IT WAS PUSHED TO A NARROWER WIDTH THAN SHOWN.

removed; second, because the water had a shorter distance to go; and third, because, other things being equal, the water would get away twenty-five times as fast. After a few weeks of operation in this manner, the work was stopped temporarily, which gave an opportunity to examine the deposited core material. After only a few days it became hard enough to walk on, many times more solid than ordinary core material.

In one other respect this coarse-grained puddle differed from ordinary puddle. There was a small relative range in its grain sizes. The fine particles were eliminated, and the volume of puddle produced was reduced, but there was no very great change in the quantity and character of the coarser particles. These coarser particles constituted the whole mass of the coarse-grained puddle. With ordinary puddle, they were mixed with a large additional volume of fine-grained stock. The grains in the coarse-grained puddle were thus more nearly of the same size, and it followed that, other things being equal, the percentage of voids was greater. This was found to be the case in the test at San Pablo, and after the material had become hard and stable, it had a percentage of voids that would have indicated lack of stability in ordinary stock. This is an illustration of the fact that voids used as an index of stability can only be properly compared for the same kind of stock.

If this method of increasing the grain size of core material was to be followed throughout the construction of a dam, obviously there would be two results. First, a great deal of fine material would be wasted. The quantity would depend on the quantity of such material in the stock used. In the natural course of events most of this would be deposited harmlessly on the bottom of the reservoir. Second, a dam built in this way would be much more stable but theoretically not quite water-tight. There would be an appreciable amount of seepage through core material of this grain size.

The amount of seepage loss may be estimated approximately. Assuming that the laws of flow are the same as they are in the finest sand to which experiments have extended, it was estimated that at the San Pablo Dam if the whole core material could be made 0.01 mm. and if the remainder of the dam had contributed nothing to tightness, the theoretical rate of seepage would be 280 000 gal. per day. The loss of this quantity of water would not be a serious drain on the supply,

if it adds to stability. Practically, as is known from filter experience, considering the core material as a filter, it would soon silt up and become water-tight.

As a practical proposition in dam construction, core material having a grain size of 0.01 mm. is to be accepted as sufficiently fine from the standpoint of water-tightness. This size of 0.01 mm. is not given as a precise limit. It may be that still coarser material would give sufficient tightness or that finer material would drain sufficiently. The size is mentioned because it is the size of material that was actually found possible to produce at the San Pablo Dam by reducing the dimensions of the pool to a minimum, and because other considerations suggest that it may be suitable.

The question remains as to whether a dam in which the pool and water quantities were adjusted so as to hold the effective size of core material at 0.01 mm., or at some other selected limit, and in which the core material was never permitted to become very wide, would consolidate itself as the dam increased in height to an extent that would eliminate lateral pressure of core material.

It is the writer's idea, that a dam built in this way would certainly be more stable and safer than one in which the core material contained an additional quantity of finer particles.

This thought is similar to an idea expressed by D. C. Henny, M. Am. Soc. C. E., who stated:*

"Probably the nearest approach to a perfect core of great thickness which can be hydraulicked, is one composed of fine sandy silt, such as is generally found in the arid West, having little cohesiveness, good self-drainage qualities, becoming hard and solid after a short time, and yet being, if not perfectly, at least practically water-tight."

Mr. Henny's description may be taken as an accurate description of the writer's idea of 0.01-mm. material. It certainly does not apply to 0.002-mm. material. Mr. Henny's other propositions in regard to fine core material such as is ordinarily used, are believed to be well founded.

The Use of Rock Fill.

In building hydraulic-fill dams, the débris above the solid rock has been most easily worked and extensively used. Such débris is frequently fine in grain size, and its use may result in an excess of core

* *Transactions*, Am. Soc. C. E., Vol. LXXIV (1911), p. 82.

material and a deficiency of toe material. To correct this tendency, rock fill has been added to the toes of a number of dams.

The rock fill has usually been placed as dry fill; that is to say, by blasting, steam shovel, and cars or carts. The cars or carts run on the toes of the dam and deposit material at the same time that the hydraulic process is being used to fill the interior.

At the San Pablo Dam (see Figs. 11 and 12), a similar result has been reached by handling the rock after blasting by the hydraulic method. Open flumes were used with a flow of from 15 to 20 cu. ft. per sec. A 6% slope was necessary as long as wooden flume bottoms were used. With steel flumes, equally favorable results were obtained with a 4% slope. About 3 000 cu. yd. per day were placed, on an average. Pieces of broken rock up to 1 cu. ft. in size were handled under these conditions. By blasting the hardest rock available, it was possible to get material for fill that contained relatively few fine particles, and all of the small quantity of very fine material was wasted. Large additions to the toes of the dam were made in this way.

Working a quarry with a high face, it was possible to throw down great quantities of material in one large blast, and to wash many thousands of yards into the dam from a single position of monitors and flumes. The economy resulting from operating so long in one place, as compared with moving the equipment all over the hillside to pick up scattered and relatively thin deposits of débris above the rock, turned out to be fully equal to the additional cost of rock excavation; and rock fills, built in this way, cost no more per cubic yard than fill made from débris.

On the other hand, the fill made in this way is believed to be heavier, and to have a higher coefficient of friction which gives it additional value as toe material. The practical methods of handling rock fill were worked out by Mr. Hawley and D. W. Albert, Assoc. M. Am. Soc. C. E.

Quicksand Conditions in Dams.

When a granular material has its pores completely filled with water and is under pressure, two conditions may be recognized. In the first or normal case, the whole of the pressure is communicated through the material from particle to particle by the bearings of the edges and points of the particles on each other. The water in the pores is under no pressure that interferes with this bearing. Under such conditions,

the frictional resistance of the material against sliding on itself may be assumed to be the same, or nearly the same, as it would be if the pores were not filled with water. In the second case, the water in the pores of the material is under pressure. The pressure of the water on the particles tends to hold them apart; and part of the pressure is transmitted through the water. To whatever extent this happens, the pressure transmitted by the edges and points of the particles is reduced. As water pressure is increased, the pressure on the edges is reduced and the friction resistance of the material becomes less. If the pressure of the water in the pores is great enough to carry all the load, it will have the effect of holding the particles apart and of producing a condition that is practically equivalent to that of quicksand.

An extra pressure in the water in the pores of such a material may be produced by a sudden blow or shock which tends to compress the solid material by crushing the edges and points where they bear, or by causing a re-arrangement of particles with smaller voids. An illustration of this can be seen in the sand on the seashore. Such sand, comparable to dune sand in size, is usually found to be saturated with water for a certain distance above the water level. This condition is maintained by capillarity. If a weight is slowly placed on this saturated sand, there is a slight settlement, the grains of sand coming to firmer bearings, and the weight is carried. A sharp blow, as with the foot, however, liquefies a certain volume and makes quicksand. The condition of quicksand lasts for only a few seconds until the surplus water can find its way out. When this happens the grains again come to solid bearings and stability is restored. During a few seconds after the sand is struck, however, it is almost liquid, and is capable of moving or flowing or of transmitting pressure in the same measure as a liquid.

Fine-grained sand in which this condition exists is called quicksand. The properties of quicksand are well known. Fine-grained sand saturated with water and then mixed with an additional 5% of water acts practically as a liquid. It will flow through small orifices or in a pipe at a 5% slope. The sand in a mechanical filter in process of washing is a typical case. When the sand is drained, in the filter, it forms a bed as hard as any sand bed. When subjected to a reverse current of water strong enough to lift it slightly, however, the volume is increased by perhaps 5%, and it becomes liquid. An object can

FIG. 11—EXCAVATING ROCK AT SAN PABLO DAM BY HYDRAULIC JET, AFTER BLASTING.

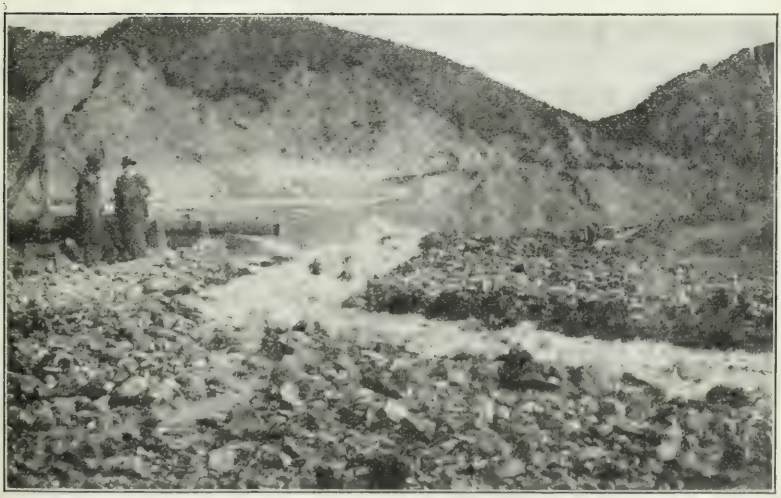


FIG. 12.—PLACING ROCK HYDRAULICALLY IN SAN PABLO DAM, FRAGMENTS UP TO 1 CU. FT. BEING HANDLED BY THIS METHOD.

then be pushed through it with scarcely more resistance than would be offered by a liquid of high specific gravity.

The conditions that control the stability or lack of stability in quicksand may also control the stability or lack of stability of materials in dams.

The puddle clay core of an hydraulic-fill dam is physically like quicksand, but with particles one hundred times smaller in diameter and a million times smaller in weight. It has the instability of quicksand in full measure and it retains it for a long time, or perhaps indefinitely.

With the coarse-grained part of an hydraulic-fill dam, that is to say, with rock toes, there may be also, at times, a similar condition. Generally speaking, it would be expected that such coarse-grained materials would have sufficient drainage to let out surplus water and prevent the possibility of an excess sufficient to destroy its stability. With hard-grained materials from glacial drift of New England and the Northern States, it is hard to conceive of a lack of drainage in gravel that would permit the accumulation of an excess of water. With the softer materials of the Pacific Coast, however, the conditions may be different. In the first place, these soft rocks by partial crushing under pressure produce fine material which tends to fill the remaining spaces and to reduce the drainage capacity.

On the other hand, each increment of loading applied to soft-grained material produces a certain compression and settlement; and with it a reduction in voids. This may happen to a toe of soft rock on the up-stream side of a dam against which water is being stored during construction. There is first an open condition with ample voids and ample drainage. As the dam is built higher, pressure increases; there is compression and reduction in porosity. Each additional increment of loading and compression means that a certain quantity of water representing the difference between the old voids and the new voids must be expelled. As long as the material remains sufficiently pervious to carry off this excluded water, the process of compression is harmless. The surplus water is pushed back into the reservoir and stability is retained. There may come a time, however, when the compression goes forward so rapidly that the surplus water cannot be carried off fast enough. When that point is reached (if it is reached), there will tend to be an excess of water in the interstices

and that excess will transmit some of the pressure that was before carried only by the bearings of solid particles, and the frictional resistance of the material will be less, and perhaps much less, than it was before.

The thought has occurred to the writer, in looking at the material that slid in the Calaveras Dam, that something of this kind may have happened on a large scale—800 000 cu. yd. of fill flowed for a brief space, and then became solid. It was, in fact, so solid that in examining it afterward, by samples and by borings, it was difficult to see how the material could have flowed—as it certainly did flow.

It may be that after the first movement there was some readjustment of the material in the toe which resulted in producing temporarily this condition of quicksand, and which destroyed for a moment the stability of the material and facilitated the movement that took place.

This will not account for the initial movement; but the initial movement of some part of the material might result in accumulating pressure, first on one point, and then on another, successively, as the early points of concentration were liquefied and in that way a condition comparable to quicksand in a large mass of material may have been produced.

Summary.

To summarize briefly the points that the writer has attempted to make and to apply them practically to dam construction:

1.—It is not well to build an hydraulic-fill dam of material of which any large percentage consists of clay or of particles less than 0.01 mm. in diameter; and in general all such smaller particles may well be wasted and excluded from the dam.

2.—By reducing the construction pool to a minimum, and by controlling it and the quantities of water used for sluicing, the core material may be held to a certain degree of coarseness by wasting all smaller particles. An effective size of 0.01 mm. may reasonably be sought.

3.—To study by borings the actual consolidation of the material, and to adjust the construction of the upper parts of the dam to the demonstrated condition of that which lies below.

4.—To make the toes large enough to resist with an ample factor of safety the whole pressure of the core material as a liquid until there

is demonstration of the solidification of the core to a point where horizontal pressure is eliminated.

5.—To increase the weight and solidity of toes by the use of rock fill, placed hydraulically or otherwise.

6.—Stability is increased by compactness. It is worth while to watch voids closely, and to make every effort to hold them at a minimum. The extra weight is advantageous, but security against compression and re-arrangement with resulting temporary quicksand conditions, can be best reached in this way.

A strict application of these principles may reduce the number of hydraulic-fill dams that are built by eliminating that method of construction from consideration where the available material contains too many very small particles. It may also increase considerably the volume and cost of those that are built. It would seem, however, that following them to a logical conclusion, with such testing as can reasonably be done, will result in eliminating present uncertainties and in putting a most useful method of dam construction on a definite and safe basis.

DISCUSSION

Mr. Curtis.

C. E. CURTIS,* M. AM. SOC. C. E. (by letter).—This interesting and instructive paper dwells on two important elements in hydraulic-fill dam construction, core material and toe material, discussed with greater care and in greater detail than the writer has yet seen. The summary is valuable because it states, briefly and clearly, important facts well worth consideration by any one engaged in building hydraulic-fill dams. It makes available, in a few sentences, facts that otherwise might require much reading and investigation to obtain. The important points are brought to the reader's attention and the things necessary to bear in mind for a successful dam built by the sluicing method are indicated.

Two important features to be carefully considered in every hydraulic-fill dam are the fine, plastic, primarily impervious, unstable core and the coarse, stable, pervious, supporting outer embankment, spoken of by the author as the toe. It is essential to have a water-tight core and equally essential to provide sufficient coarse material to hold the core in place and give stability. The whole secret of hydraulic-fill dam construction, in the writer's opinion, is the proper handling of these two features without making either excessive and, at the same time, obtaining the desired result, namely, a tight and safe structure. The details of handling these features are, of course, necessarily different in every operation, because of the varying conditions, but the principle is the same always.

The writer was much interested in the description of the pipe tests used to determine the solidity and character of core material, because of the similarity in both methods and results to those obtained in his own experience. He found that 30 or 35 ft. was about the limit that two men could push a 1-in. pipe down into the soft material of the core, and that 60 ft. was about the limit the same pipe could be driven by using hand hammers. Samples taken from the pipe were quite uniform and the most satisfactory of any obtained. There was always the feeling, however, that a sample obtained in this way was more compact, due to the manner of taking it, than the material really was in the dam, but samples so taken are the best that can be obtained, as far as the writer's experience goes.

The author states that he found it necessary to stop sluicing in order to make pipe tests. The writer has known of instances where such tests were usually made while sluicing was in full progress, without any trouble or inconvenience, the work being done by men on a raft.

In a locality where fine material predominates and it is difficult to get coarse material in large quantities, it might become necessary

* Ithaca, N. Y.

to waste the very fine particles of the core material in order to secure compactness, but, on the other hand, in a locality where coarse material predominates, as it does in some places, it is perhaps not always necessary to do this, but the finer the core material the more necessary and difficult it becomes to hold the fine particles where they belong. It can be done successfully only by providing an adequate mass of coarse material well and carefully placed. Mr. Curtis.

Some years ago the writer was in charge of an hydraulic-fill dam where great care was exercised to retain all the fine material possible in the core, as it was thought to be necessary for tightness and would also add to the volume in the embankment. This dam, more than 90 ft. high, was built without any slips or constructional mishaps and is unusually tight with little if any seepage through it. There was, however, an abundance of both coarse and fine materials advantageously located for sluicing so that the proportions could be easily controlled. The work referred to is the Quemahoning Dam of the Cambria Steel Company, near Johnstown, Pa.

Fig. 13 shows a cross-section of the completed dam with the general sub-divisions or zones of material indicated approximately as the dam was built. The late Frederic P. Stearns, Past-President, Am. Soc. C. E., was the Consulting Engineer for the dam; Joseph Morgan, Consulting Engineer for the Cambria Steel Company, was Consultant on the dam and the pipe line; and John W. Gocher, Chief Engineer of the Cambria Steel Company, was Chief Engineer of both. The writer had charge of the construction of the whole development including both the dam and the pipe line. Fig. 13 contains several brief statements of facts pertaining to the dam, and the following notes in regard to the pipe line, which was built at the same time and is a part of the development, are pertinent.

The distance from the dam to the works by pipe line is 14 miles; the pipe is 5 ft. 6 in. in diameter and its total weight is 13 000 tons; it crosses Stonycreek River thirteen times, passes through five tunnels aggregating 9 000 ft. in length and the daily flow is from 75 000 000 to 90 000 000 gal. The catchment area is 92 sq. miles and the flooded area 900 acres.

This dam, 93 ft. high above the natural surface of the valley, was built with flat slopes of 4 to 1 for the lower and 3 to 1 for the upper half of the embankment; both the up-stream and down-stream slopes were the same. The width of the spillway is 250 ft., the maximum calculated flow over which is 11 800 000 000 gal. per 24 hours. The dam is 950 ft. long on top and contains about 600 000 cu. yd. of material, of which more than 500 000 cu. yd. were sluiced. The two dikes (marked *d*) one at each toe, were built by team and scraper of material taken from the middle third of the area covered by the dam. A certain limited quantity of material on the top was placed by hand

Mr. Curtis.

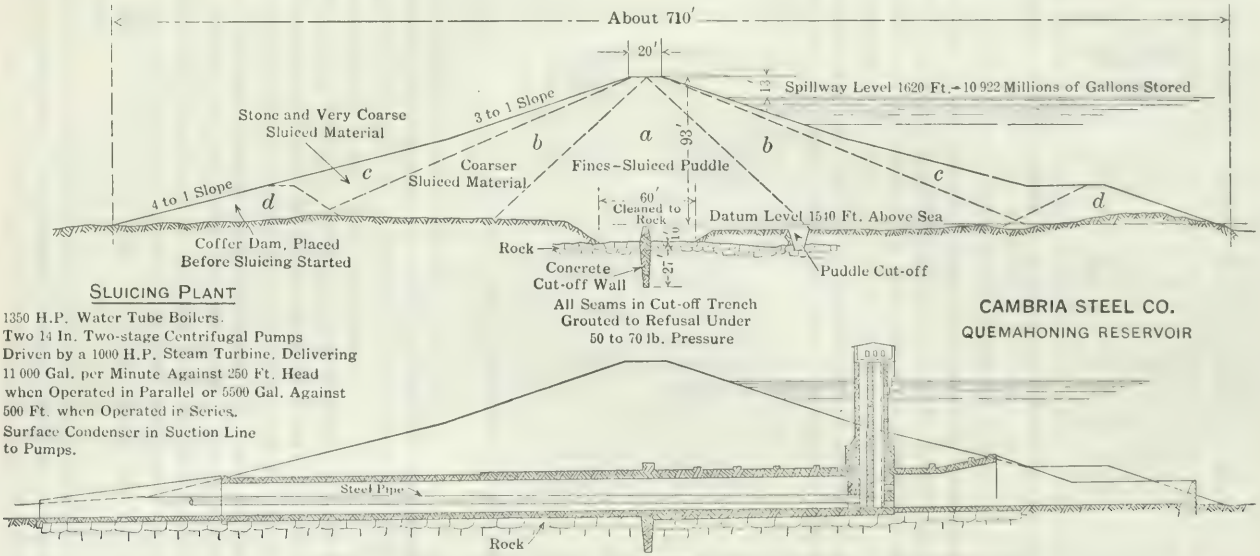
in the usual manner, all other material having been sluiced. The sides of the valley were about 200 ft. higher than the top of the finished dam, so there was sufficient elevation to secure the desired grade for sluicing and, at the same time, discharge all the material on the dam without rehandling. This condition made sluicing comparatively simple. The largest rock carried by the sluices was about 1 ft. square by 3 ft. long, and a depth of water of between 6 and 8 in. was sufficient to transport rocks of this size on a 6% grade. One side of the valley was shale rock in ledge formation with only from 4 to 6 ft. of glacial drift on top, the other side being composed of a heavy deposit of glacial drift and clay with many stones and boulders mixed through it. It was not difficult, therefore, to get nearly any desired proportion of coarse and fine material.

The shale was drilled and shot with a light charge of dynamite so as to shake it up a little without throwing it down. This made sluicing easier and, by so doing, it was quite practicable to carry a 40-ft. vertical face in the sluicing pit. It also hastened the work to shoot the clay. In either material, a very small crack was all that was necessary to give the water a chance to do its work.

All material was conveyed to the dam in wooden sluices, 24 in. wide and 18 in. deep, built on a 6% grade, and placed so that they discharged the material at the extreme outer edges of the embankment on both up-stream and down-stream toes. The very coarse material, composed of large stones and rock, remained where it was discharged from the sluice-box, while the intermediate grades, between very coarse and very fine, were deposited at varying distances from the sluice, depending on the quantity of water used in sluicing and the elevation of the central pool for controlling the deposit of core material. The fine sands produced by the sluicing would build up on a very steep slope at the edge of the pool for a height of 2 or 3 ft., then slough off and slide into the soft core material. This operation was repeated to a greater or less degree during the whole sluicing period. The central pool was usually from $1\frac{1}{2}$ to 2 ft. deep, and at these depths it was large enough, for two-thirds of the height of the dam, to allow quite complete sedimentation so that there was very little loss over the waste weir provided to carry off the sluicing water.

The width of the core at the base was a little less than one-third the total width of the section, decreasing as the height increased. The proportions gave room for a sufficient bulk of coarse material to hold the core in place. It is the writer's opinion that this thickness of the core is greater than was necessary and in the future he would be inclined, under like conditions, to reduce its cross-section materially on the theory that less thickness is ample for water-tightness and that the increased quantity of coarse material is an added security against sliding. Under some conditions and in some locations, depending

Mr. Curtis.


About 710′
20′
Spillway Level 1620 Ft.—10 922 Millions of Gallons Stored
Stone and Very Coarse Sluiced Material
3 to 1 Slope
4 to 1 Slope
Coarser Sluiced Material
Fines—Sluiced Puddle
93′
60′
Cleaned to Rock
Datum Level 1540 Ft. Above Sea
Coffer Dam, Placed Before Sluicing Started
Rock
Concrete Cut-off Wall
57′
Puddle Cut-off
All Seams in Cut-off Trench Grouted to Refusal Under 50 to 70 lb. Pressure
SLUICING PLANT
1350 H.P. Water Tube Boilers.
Two 14 In. Two-stage Centrifugal Pumps Driven by a 1000 H.P. Steam Turbine, Delivering 11 000 Gal. per Minute Against 250 Ft. Head when Operated in Parallel or 5500 Gal. Against 500 Ft. when Operated in Series.
Surface Condenser in Suction Line to Pumps.
CAMBRIA STEEL CO.
QUEMAHONING RESERVOIR
Steel Pipe
Rock

Concrete Couduit, 30 Ft. Wide 20 Ft. High, for Passage of Stream while Dam is under Construction; Upper End Afterward Closed by Gate-house and Pipe Laid Through it.
13 250 Cu. Yd. Concrete in Conduit.

FIG. 13.

Mr. Curtis. largely on the kind of material, it might be best to steepen the slopes, omitting the 4 to 1, for instance, and using 3 to 1 for the full height of the dam. A steeper slope than the latter does not seem to be desirable.

In support of this statement the writer will mention the Salt Lick Dam, of the Johnstown Water Company, which has 3 to 1 slopes both up stream and down stream. It was built at the same time and in the same manner as the Quemahoning Dam. Frequent pipe tests were made of the core, as the work progressed, in order to obtain some idea of the solidity of the material. As mentioned previously, from 30 to 35 ft. was the average depth reached without driving, and 60 ft. by driving. At corresponding depths the samples were usually the same and about the consistency of stiff putty.

No tests were made for fineness of the core material, but chemical analyses and determinations of specific gravity, the results of which are given in Table 3, were recorded.

TABLE 3.—Analysis of Core Material, Quemahoning Dam.

No.	Percentage of loss on ignition.	Percentage of iron peroxide.	Percentage of alumina.	Percentage of silica.	Percentage of calcium carbonate.	Percentage of magnesium carbonate.	Specific gravity.
39	8.06	7.08	17.36	61.62	Lime	3.10	2.45
40	7.70	8.34	13.08	68.52	0.68	1.15	2.63
41	6.99	8.24	14.68	67.56	0.76	1.41	2.55
42	5.16	5.96	10.84	76.52	0.36	0.68	2.44
43	7.14	6.96	13.44	70.52	0.60	0.82	2.50

It should be noted that the specific gravity of the material varies as much as 8 to 9%, and is less than the result given by Mr. Hazen, which was 2.65.

It may be of interest to give the results of the analyses of two samples of sluiced material from the Sánta Amoro Dam, São Paulo, Brazil, sent to the writer by the late James D. Schuyler, M. Am. Soc. C. E., in 1918. The analyses are given in Table 4.

TABLE 4.—Analyses of Sluiced Material, Sánta Amoro Dam.

No.	Percentage of loss on ignition.	Percentage of iron peroxide.	Percentage of alumina.	Percentage of silica.	Percentage of calcium carbonate.	Percentage of magnesium carbonate.	Specific gravity.
21	13.82	10.85	36.08	38.68	None	0.22	3.94
22	14.70	8.68	37.80	37.20	"	0.17	3.90

Mr. Curtis.

There is a marked difference between the analyses of the material in the Quemahoning Dam and that from South America. The extremely high specific gravity of 3.90 and 3.94 is, the writer assumes, rather unusual, although sluiced material from the Nuunau Dam, Honolulu, Hawaii, showed a specific gravity of 3.11. The writer is indebted to Mr. Schuyler also for this sample.

The solidity and compactness of the Quemahoning Dam may be indicated by the amount of settlement after completion. To provide for any possible shrinkage due to settlement, an additional height of 3 ft. was added to the dam. Careful levels were taken frequently along both the up-stream and down-stream edges of the top width and after a period of nine months from the date of completion, the average settlement was not more than 9 in., or less than 1% of the height. From this, it is apparent that although the core material was very soft when placed, it later became a rather compact mass of plastic clay. It is quite probable that the dry material, placed by hand on top, settled into the semi-liquid layer of core material, absorbing the water it contained, so that the whole mass became stable.

The writer has referred at some length to his own experiences with hydraulic-fill dams because the results obtained coincide so well with, and support, the author's statements. The Quemahoning and Salt Lick Dams show that dams of considerable height can be built successfully by the hydraulic-fill method.

The fineness of the core material and the size of particles that should be wasted depend very largely on the character and quantity of coarse material available. It would seem to be desirable to sluice as much of the coarse material as possible and deposit it at the outer edge of the embankment. In this way the medium grades of material are carried toward the central pool by the water, becoming thoroughly compacted in the operation, and making a firm unyielding mass with practically no settlement after completion. Throughout the construction of the Quemahoning Dam, all the area from the outer edge to the central pool was firm enough to walk on without sinking in.

From what the writer has seen elsewhere, and from his own experience, he is satisfied that where conditions warrant its adoption, hydraulic-fill dam construction is practical, economical, and a very satisfactory type of earth dam. It is a type of construction which, although apparently simple, requires close attention to details.

Mr. Chase.

Richard D. Chase,* Assoc. M. Am. Soc. C. E. (by letter).—After spending about two months at the San Pablo Dam, the writer became so impressed with the small labor and plant cost, that he thoroughly believes the hydraulic-fill method is worthy of being adopted in the East. It is perfectly true that failures of these dams have occurred, but the number of successful ones is sufficient to indicate that success

* New Bedford, Mass.

Mr. Chase. is not due to chance, and Mr. Hazen gives good theoretical and practical reasons for the failure or success of such dams.

When one observes the light soil and the soft, friable, quickly disintegrating rock which are the only available materials at San Pablo and Calaveras, one can realize that the hydraulic-fill method would be most applicable to the heavy glacial drift of the northeastern part of the United States.

With the present unsatisfactory labor market, particularly for heavy manual labor, any method of construction that will decrease the number of men on a piece of work deserves more than usual consideration. It was hard to believe that the half dozen men on the dam and the few on the hillside were putting in 100 000 cu. yd. per month. There was also a comparative absence of expensive plant. It is true the situation was most favorable, with the borrow-pits above and directly adjacent to the ends of the dam—an advantage hardly probable in the East—but even with preliminary handling the advantages are so obvious that serious consideration should be given to the hydraulic-fill method for any earth dams.

The writer believes that this paper will clarify the theory and practice of the hydraulic-fill dam so that it will not be treated as a doubtful experiment, with the chances of failure outweighing its obvious advantages, and that by following the theory laid down one can feel that he is proceeding along sure and definite lines, with a method provided for testing the results obtained so that the chance of failure is eliminated.

Mr. Elliott. G. A. M. Elliott,* M. Am. Soc. C. E. (by letter).—This paper is timely in that it brings to a head and assembles, where it is readily accessible, the results of present experience in hydraulic-fill dam construction. The fundamentals of this type of dam, so clearly set forth by the author, have too often received little consideration. This method of dam construction is attractive on account of its comparative cheapness, and has been adopted for that reason in some instances, when, with the same materials, another construction method would have been more suitable. The hydraulic-fill dam is not to be regarded as a general type, to be built under all conditions of earth-fill, but should be used only when all the facts make it advisable.

It seems to the writer that if the proper relation between width of toes and core is maintained, the grain size of the core material will automatically take care of itself. For instance, if the material used is of a hard rock base, such as a gravel deposit, it will probably not break up under handling sufficiently to give a preponderance of very fine core particles, and naturally a narrow core and comparatively wide toes will result. An example of this is to be found in the Paradise Dam near Chico, Cal. The material in that locality is of volcanic

* San Francisco, Cal.

origin, and some difficulty was experienced in obtaining a sufficient quantity to insure an impervious center. During construction the drainage through the toes was of such proportions as to be alarming to one not familiar with the hydraulic method. This dam has been completed and in use for three years with excellent results so far as tightness is concerned. In the case of softer materials which, in handling, will break up to such an extent that an excessive quantity of fines is delivered to the "core-pond", the wastage of the surplus puddle necessary to maintain a narrow impervious center will result in automatically discarding very fine particles. A core-pond width of not more than 20% of the total thickness of the dam at all elevations is sufficient for water-tightness, is not too narrow to interfere with ease of construction, and, with proper care in distribution, there is no danger of tongues of coarse material encroaching on the impervious area. To hold to this width in dams built with the comparatively soft materials which must sometimes be used, will necessitate wasting considerable material. This increases the construction cost. It is difficult to persuade engineers of the necessity for wasting material which has cost a great deal of money to break down and transport a considerable distance. Until this fact is fully recognized, failures of hydraulic-fill dams will continue to occur. Mr. Elliott.

It is important not to exceed the predetermined boundaries of the core-pond at all stages of construction. When the fill is carrying an excess of core material there is a tendency to retain all of it, on the theory that even though the clay overlaps the dry toe area, it can be displaced later by the use of other material carrying an excess of rock. In some cases this displacement is attempted by dumping dry fill along the edges of the pond. At best, it is uncertain just what happens under this treatment. It is the writer's opinion, after some experience with the latter method, that although, of course, the dry material does displace a certain quantity of liquid clay, the character of the resulting fill is not as good as the dry banks laid down by sluicing. If clean rock is used, the interstices are filled with liquid clay, and the hydrostatic pressure may be transmitted through the new fill to the original dry banks. The new fill is lubricated, and if the internal pressure is sufficient to move the dry toe, the mass of lubricated rock, instead of adding stability, will move almost as readily as the core. If a mixture of earth and rock is used there may be danger of masses of this material trapping large pockets of clay, which, in effect, provides points of lubrication. Well-drill explorations of an embankment made by combined hydraulic placing and dry dumping will not give complete data, as the operation of boring the well grinds up the fill into small particles. Even if this character of exploration does indicate that the coarse material has apparently displaced the clay, it tells nothing in regard to the fact that the voids in the rocks may be filled with liquid core.

Mr. Elliott.

Exploration by borings in the up-stream toe of the Calaveras Dam made during construction indicated a solid fill in locations in which the core had been replaced by dry fill. In the light of later events, the writer is convinced that the mass of coarse fill, placed by wagon in and along the edge of the core-pond, was permeated by the semi-liquid clay core, and that it added little or nothing to the stability of the structure. The action of the down-stream toe, which, although not as thick as the combined up-stream toe (placed by hydraulicking and wagon), was built of dry material dumped from cars on trestles, bears out this conviction. This embankment was built up vertically, that is, the inner face did not encroach on the core-pond to any great extent with increased elevation, and the material was not placed in the core itself for the purpose of forcing it back. Under this method there was no opportunity either for trapping the clay masses or for the voids in the dry bank to be filled with wet core material. In spite of its smaller section, this embankment did not move horizontally. Mr. Hazen's quicksand theory would apply directly to that portion of the dry embankment built by displacing the core by dry dumping.

A semi-hydraulic-fill method has been used with success on the Pacific Coast in cases where the material available consisted of soft rock which would not lend itself to economical spreading and rolling and would break down badly with water transportation. Briefly, the material is transported in cars and dumped on the up-stream and down-stream embankments, leaving a predetermined distance between for the core. This space is filled with water, and a pump mounted on a barge is used to jet down the exposed inner surfaces of the banks. By using this plan the core is under absolute control at all times. The dry banks are loosely placed, and consolidation occurs in time through natural settlement. In the case of small dams which might be built in one season and filled immediately thereafter, there might be some objection on the score that the embankment had not consolidated sufficiently, but on large dams requiring several years to construct the settling action due to rain and time would offset this feature. The down-stream embankment of the Calaveras Dam was built in this manner, and showed no signs of settlement beyond two years after placing. The Haiwee Dam built by William Mulholland, M. Am. Soc. C. E., in Owens Valley, California, was successfully constructed in this way.

Mr. Howells.

J. M. Howells,* M. Am. Soc. C. E. (by letter).—Engineers are indebted to the author for an instructive analysis of the fundamental principles involved in hydraulic-fill dam design and construction and as a contribution to the discussion, the writer presents only minor variations of viewpoint from those taken by Mr. Hazen. One of these differences is in the matter of homogeneity of core material, for the

* San Francisco, Cal.

writer has noted that almost invariably toward the finish of a dam, as the top approaches the form of an elevated canal, the construction water flows so rapidly that it carries off core material needed for water-tightness. Also, at this stage of the work, as the hydraulic fill is delivered at both the up-stream and down-stream faces, it has been difficult to prevent the deposit of coarse porous sand "tongues," which often reach entirely across the central pool, thus tending to loss of water-tightness. Mr. Howells.

The writer wishes to present one phase of Mr. Hazen's paper concerning which confusion on the part of the members of the Society seems to prevail, that is, the simple question as to what constitutes an hydraulic-fill dam.

At a meeting of the San Francisco Section of the Society, a committee was appointed, composed of Mr. Hazen, C. E. Grunsky, M. Am. Soc. C. E., and the writer, as Chairman, to "formulate and present to the Section, the definition of the term 'hydraulic-fill dam' ".

As this type of dam is a Western development, it was considered appropriate that the San Francisco Section should presume to appoint a committee to formulate this definition. The report of that committee follows:

"*Definition of an Hydraulic-Fill Dam.*—The hydraulic-fill dam method of constructing earth dams implies a conveyance of material to its place in the structure by flowing water, but the material may be brought to the structure by mechanical means, providing it is properly washed and segregated while it is in the process of being deposited.

"Any dam constructed throughout by this method is an hydraulic-fill dam.

"The method further implies more or less separation of the material into two distinct classes, each deposited in definite locations within the cross-section of the dam—the coarsest, most gritty, and porous conveyed and deposited by running water and constituting the outer portions of the dam, to secure stability; and the finest and most water-tight material deposited under standing water and constituting the central portion, insuring water-tightness.

"Whenever an appreciable part of a dam is built by the dry-fill method and a part by the hydraulic-fill method, the structure is no longer to be classed as an hydraulic-fill dam, but is a combination structure—a loose rock dam with hydraulic-fill core, a dry-fill earth dam with hydraulic-fill core, etc., as the case may be."

According to this definition, the Calaveras Dam is not an hydraulic-fill dam.

In referring to the mysterious action of the material in the toes of the Calaveras Dam, its lack of stability before the accident, and and its tendency to stabilize after the accident, one might look to a fact not brought out clearly enough by the author, namely, that, as previously stated, the Calaveras Dam was in no sense an hydraulic-fill dam. It was started as such, and about 60 ft. of the base was built in

Mr. Howells.

that manner, and this part was found to be intact after the disaster. It is important to bring out this fact, as many of the technical papers have called this structure an hydraulic-fill dam.

Above the 60-ft. base, the dam was composed of a pumped hydraulic-fill core of slimes, built between toes, like dry railroad dumps, with no washing of the latter to carry away the native slimes which when wet are inherent in their masses. Neither was any apparent effort made to build these dry fills in thin horizontal layers, a practice considered as standard for earth dams other than those built by the hydraulic method. If there were any layers in these toes, they were on a slope of about $1\frac{1}{2}$ to 1, and resulted from the sliding over-dump of the dry material. As this dumping of the toe material progressed, great porosity or looseness of the mass resulted, and the writer has been told on good authority that the puddle-core material often found an outlet by irregular channels through this loose dump to the faces of the structure. Thus, while settlement was in progress, lubricated planes were being arranged to take part later in the ultimate disaster.

That the slip did not take place on the down-stream face which was steeper and theoretically less stable, was probably due to its opportunity for drainage. On the up-stream face, both the reservoired water and concrete facing interfered with such drainage. This, together with the sloping structural planes of deposit (already in movement as they successively settled), must explain to a great degree the point mentioned by the author in his endeavor to account for the low coefficient of friction in otherwise fairly stable material.

Had the Calaveras Dam been a true hydraulic-fill dam, its toes would have been brought up in thin vertical increments from which all slimes would have been washed and stability insured.

The writer agrees with the author on the possibility of too broad a core being left where the true hydraulic-fill section ended, and that this might add to the tendency of the face to slide or to transmit pressure dangerously close to the faces; for in an hydraulic-fill dam of the height proposed, stability could be insured only by great width of base of both toes. This is shown by the accepted formula in which coefficient of friction and weight of mass rather than percentage of voids controls.

In this connection there is grim comfort to the builder of hydraulic-fill dams or any of their variations, namely, that they are never so weak as during construction, for at that time they are virtually two slender dams storing between them a heavier-than-water fluid. Therefore, if they stand under these conditions, they have been tested with a liberal factor of safety and are ready to assume their greater responsibility of storing a great volume of water.

As for the author's suggestion to waste core material finer than a given size: It often happens that this fine material is needed as against the porosity of the river bed itself above the dam, and in cases of dam

construction within an area of about 200 000 sq. miles in the Northwestern United States, where porous lava flows cover almost the entire country, this waste would be useful. The Big Meadows Dam is within the area mentioned, and its successful construction and seven years of operation argue for the special adaptability of the hydraulic-fill dam to such foundation conditions. Mr. Howells

The author places a great deal of stress on his void theory and, within the limitations which he places around it, there is no doubt but that the percentage of voids in the puddle core can be studied profitably, but, after all, no reliance should be placed on the puddle core for stability. It has another office.

As to the toes of hydraulic-fill dams, voids are desirable rather than otherwise, and should add to rather than detract from the stability of the entire structure.

In the original conception of the hydraulic-fill dam it was hoped, more often than later proved feasible, to secure a gradation of material from coarse at the faces to fine in the center, similar to that described in the paper* by the late James D. Schuyler, M. Am. Soc. C. E. This would insure greater voids toward the faces, better drainage, and less chance for water pressure as the faces of the dam are approached, and any drainage from the puddle core, or perhaps through the dam after construction, loses the possibility of destructive pressure by successively entering more and more porous beds of sand, gravel, and rock. Thus, the pressure diminishes and drainage flow begins so far within the structure as to be inert as against destructive effect. This was noticeable for its absence in the railroad-dump type of toes exemplified in the Calaveras Dam.

As to the use of rock-fill in the toes, this, at times, seems to be excellent practice and became a part of the design of the Big Meadows Dam, in the construction of which the problem of stability and water-tightness presented itself in a peculiar form. That is, it was necessary to connect the central puddle core with a water-tight layer placed over the stream bed for approximately $\frac{1}{4}$ mile up stream from the dam. By first placing massive basalt rocks for stability into the proposed up-stream toe, on a gritty foundation of native boulders, gravel, and sand, it was possible later to pass fine hydraulic fill through this basalt rock-fill and onward up stream to line the old river bottom and also to secure connection of this fine river lining with the puddle core of the dam.

In conclusion, the writer wishes to state that he agrees generally with the author's final summary.

D. C. Henny,† M. Am. Soc. C. E. (by letter).—The author has given with his usual clearness the reasons which render the use of very fine material in the core of hydraulic-fill dams objectionable. Dams Mr. Henny.

* *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 196.

† Portland, Ore.

Mr. Henny.

with such cores, however, have been built and have not failed, so far as the writer is aware, after construction had been carried to completion.

An interesting and perhaps important question is the condition of the core after several years' use. The author points out that a reduction of water content from 70 to possibly 50% may proceed by vertical settlement with relative promptness. There comes a point, however, when the movement of water through the mass becomes extremely slow, while a condition of horizontal pressure transference and lack of stability still exists. Full stability may be reached after voids have been reduced to about 40 per cent.

It was pointed out that a void reduction from 50 to 40% means that 6 cu. yd. would become compacted to 5 cu. yd. Whether such a core in a dam which periodically or permanently stores water, will ever reach this degree of compactness may well be doubted, as the repeated watering which the core receives, will permanently prevent complete consolidation on the water side, and this retarding effect may be carried far into the body of the core.

It is quite probable, however, that the down-stream side and the upper portions of the core will undergo a drying out process through slow evaporation, and thus it may be expected that the core, especially near the top, may show a condition of supersaturation on the up-stream side below the level of high water, gradually changing in a down-stream and upward direction to one of compactness.

The compacted or dryer portion must have lost volume. The resultant condition in the core of a dam like the Necaxa has never been examined by shafts, so far as the writer knows. Volume reduction may have taken place through vertical settlement, but after the water content has been reduced to some limit like 50%, it is not improbable that further drying will produce vertical cracks, and such cracks in themselves would act as drains and thereby expedite the drying process.

If a dam with an hydraulicked clay core was to store no water for a long series of years, it is conceivable that cracks might reach through the entire core, at least in the upper portion of the dam, a condition which if not immediately dangerous in view of the high power of resistance to erosion which dry compacted clay possesses, is neither desirable nor permanently safe. Such a condition may be regarded as another important objection to the use of clay in cores, at least until actual inspection of clay cores in existing dams proves the foregoing conception to be erroneous.

The writer wishes further to call attention to the effect which the rise of water against a dam with a clay core, either during the construction period or thereafter while the core is still semi-fluid, may have on the possibility of a slide toward the reservoir.

Mr. Henny.

If Fig. 14 represents the cross-section of a completed dam, of the dimensions indicated, with a semi-liquid core, and the weights of material may be assumed to be similar to those found by the author at the Calaveras Dam, as follows:

Weight of core material with about 50% water content	100 lb. per cu. ft.
Weight of material in outer sections.....	120 lb. " " "
Weight of material in outer sections when dry	110 lb. " " "
Weight of material in outer sections submerged	73 lb. " " "

then, with water in the reservoir at or below the base, the weight on the base, *AB*, of the up-stream toe section per linear foot of dam is 400 tons. The horizontal outward pressure of the core is 202 tons and the sliding factor is approximately 0.5.

As the water in the reservoir rises there will be created a horizontal counter pressure tending toward a reduction of the sliding factor. At the same time, however, the weight on *AB* will become reduced through submergence of the material, tending to increase the sliding factor. The combined effect is shown for various heights of reservoir water to the right in Fig. 14.

It will be noted that the danger of sliding increases 30% as the reservoir fills to 40 ft. above the base of the dam, after which it decreases. This indicates the danger which may exist in allowing the reservoir to fill during its construction or even some time after its completion.

In the case considered, with reservoir empty, a sliding factor of 0.50 is calculated in a dam with a water slope of 3 to 1. The author argues that such a sliding factor will be obtained with a water slope of 2 to 1.

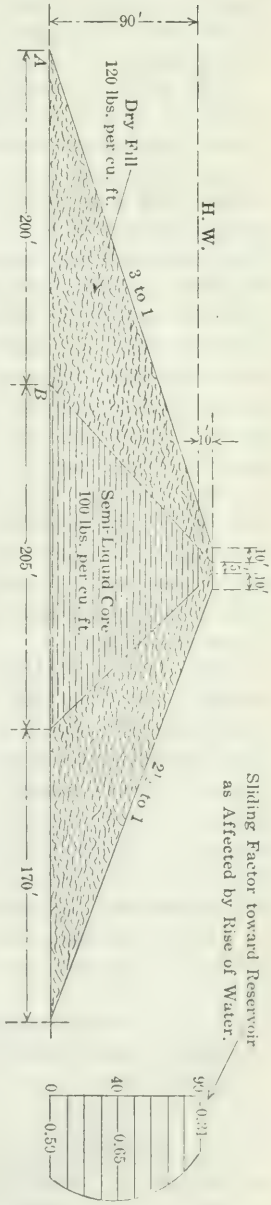


FIG. 14.

Mr. Henny. This is, however, under essentially different conditions. The difference lies partly in that Mr. Hazen bases his statement on equality of weight of toe and core material instead of the weights of 120 and 100 lb. per cu. ft., respectively, herein assumed.

The principal difference, however, lies in his assumption that the weight of the entire triangular mass in the dam up stream from the top contributes toward resistance to sliding. This can be true only if the core material, or horizontal layers of it, do not extend under the slope. Where this does occur, the author mentions for the Calaveras Dam sliding factors of 0.20 and 0.22, which are obviously average factors for the entire toe base and not the much higher factor in the toe material considered by itself, which, as the writer sees it, is controlling.

Reference in this connection has been made in a brief discussion of this problem* in which the writer argues, in connection with the Calaveras Dam, from such information as was available on the ground and as was subsequently furnished by the author, that at the time of the Calaveras slide the sliding factors in the base of the toe material may have been close to 0.7 toward the reservoir and 0.5 away from it. These values although they cannot be claimed to be accurate, due to lack of definite information regarding distribution of materials in the dam, at least explain satisfactorily why sliding took place in an up-stream and not in a down-stream direction, and that so far as information goes, 0.7 may have been approximately the coefficient of friction in the toe material.

The writer's analysis may be in error in so far as he assumes the weight on the toe base to be that of the material vertically above it. This may be reasonably correct for a dry-built dam, but in the case considered may be in error in view of the tendency of the entire outer section to slide toward the reservoir along the up-stream side of the core. To test this point a brief analysis was made of the case illustrated in Fig. 14. Neglecting the weight of the small part of the dam above the high-water line and considering only the part of the dam up stream from the vertical, *CD* (Fig. 15), the following values result:

Weight of toe section	618.3 tons
" " core "	222.8 "
Total weight	841.1 "
Load on core base 4.5×99	445.5 "
Load on toe base	395.6 "

The weight of toe material vertically above the toe base is 400 tons, so that in this case the previous assumption is practically correct, as is also indicated by the loading diagram in Fig. 15.

* *Engineering News-Record*, March 6th, 1919, pp. 487–488.

Mr. Henny.

This close equality, however, may be somewhat accidental. Submitting the case of the partly completed Calaveras Dam, before the slide, on the basis of the section shown in the writer's article previously quoted, to a similar analysis, the figures stand as follows:

Total weight of up-stream section	1 787 tons
Weight on core base	1 304 "
Weight on toe base	483 "
Weight of toe vertically above toe base	548 "

There is considerable difference in this instance. If the latter analysis is the more correct, it would seem that a large part of the weight vertically above the toe base is transferred to the core base; also that the coefficient of friction at the time of sliding may have been in excess of 0.7.

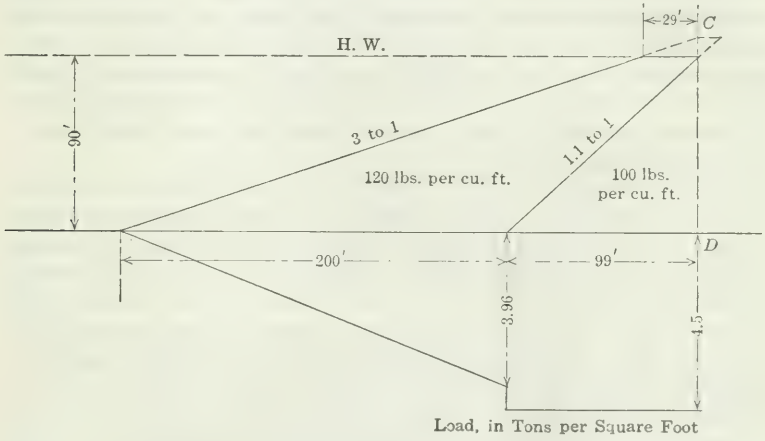


FIG. 15.

The uncertainties unavoidably involved in dealing with a structure of which the interior condition is so little known preclude the derivation of any definite friction coefficient and show the necessity of adopting some factor of safety, as argued by the author.

The author by an interesting experiment at San Pablo Dam has found coefficients of friction of 0.45 to 0.55, and for harder material of 0.70. He found further that values with pressures lower than those due to a depth of fill of 200 ft. were found to be distinctly larger, and he considers the materials at San Pablo and Calaveras as quite similar. Considering that the toe pressures at Calaveras before the slide were due to an average height of fill of less than one-third of 200 ft., there appears to be nothing in the result of the experiment disproving a possible friction coefficient at Calaveras of 0.7.

Mr. Henny.

The limitation as to fineness of material which is to form the core is obviously a measure of safety, and the author's definition of the limit is especially welcome. It is probable also that it can often be applied in construction without thereby rendering the hydraulic process uneconomical. Undoubtedly, cases will arise, however, where with an abundance of clay and intermingled coarse material a shortage of the finer particles approaching the 0.01-mm. size will occur, and that a clay core must be depended on for tightness.

Safety and economy will then combine in suggesting a far thinner core than has been commonly used heretofore. The author mentions testing out a narrow pond at the San Pablo Dam, so narrow indeed that hardly more than a muddy creek remained in the middle of the dam. The writer's experience leads him to think that under such extreme conditions the passing of coarse sand layers or lenses through the entire core is highly probable, due to changes in the material delivered from the pit—an undesirable although not necessarily dangerous result. The paddling of pond material described in connection with some dams has been practiced in the past to destroy such stratification. The writer, however, has found it to be practically impossible to mix pond material in that manner where the sand layers were of any appreciable thickness.

If a clay core must be used and sufficient graded coarse material can be sluiced to hold the core material and form the outer section of the dam, the proper solution may be a core of no greater thickness than can be hydraulicked without danger of sand layers passing through, the remainder of the clay to be wasted. The design of the Calaveras Dam is in accord with past practice, yet there appears to be no reason why a similar dam should not have been entirely tight with a core 100 ft. thick at the base instead of over 400 ft., and it is certain that with such dimensions the slide could not have occurred.

The author's conclusion that with a coefficient of friction of 0.5 in the toe material and with equality of unit weight of toe and liquid core, an outer slope of 2 to 1 will provide a factor of safety of 1, implies, as previously stated, that a liquid core in a solid mass or in horizontal layers does not extend under the slopes. If the core occupies one-third of the mass, a factor of safety of 1 under the same assumption of a coefficient of 0.5 would require a slope of 3 to 1 instead of 2 to 1, a distinction which it may be of great importance to bear in mind in view of the fact that hydraulic-fill dams, in all cases with which the writer is familiar, as well as dams with the narrowest feasible clay core, will have a considerable quantity of core material extending under the slopes.

Referring to the claims that considerations of stability require hydraulic-fill dams to be given flatter slopes than other earth dams, the writer believes that although from the previous arguments it may

be concluded that such claims are valid in the case of cores of large dimensions consisting of very fine material, they lose force where the clay core can be made thin and the material for the outer sections is reasonably coarse, and such claims appear to be entirely invalid where the core is composed of larger particles of about the limiting size of 0.01 mm. suggested by the author, since in that case the core has self-draining properties and becomes promptly as hard and stable as material in a well compacted dry-fill dam. Mr. Henny.

A. T. Goldbeck,* Assoc. M. Am. Soc. C. E. (by letter).—In his discussion of the measurement of pressures existing in the cores of hydraulic-fill dams, the author mentions a method now being used by the Miami Conservancy District, with a special type of apparatus developed by the writer with the collaboration of Mr. E. B. Smith and others connected with the laboratories of the United States Bureau of Public Roads. It would seem that the author has overlooked the possibilities of this method for, apparently, he is not fully informed as to the degree of accuracy to be expected from this type of instrument. Mr. Goldbeck.

For the benefit of those who are not familiar with the soil-pressure cell mentioned by Mr. Hazen and used very successfully during the past several years in the U. S. Bureau of Public Roads for measuring the distribution of pressures through granular materials, it may not be amiss to introduce at this time a short description of the apparatus, together with illustrations of its application and tests confirming its accuracy.

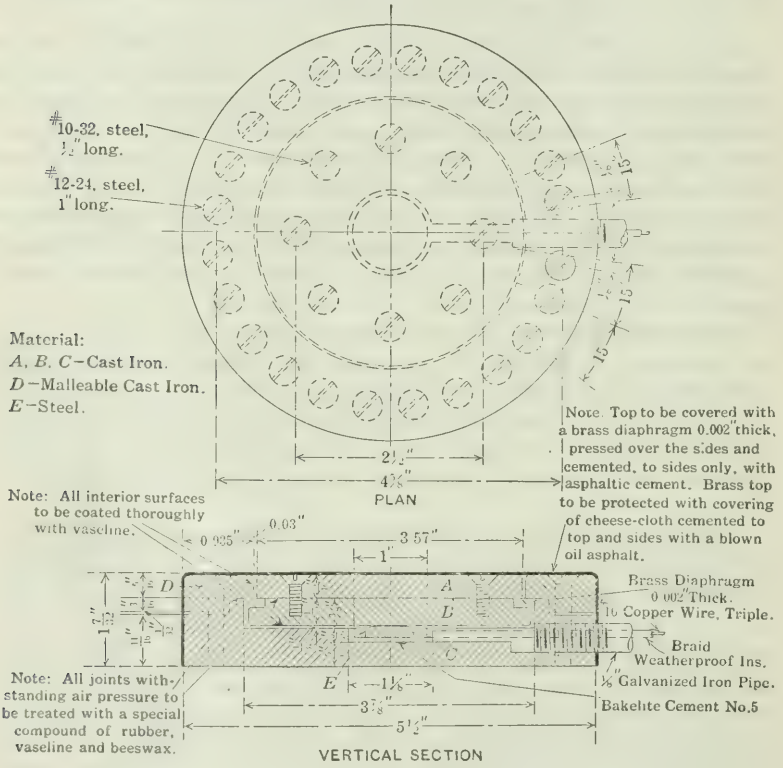
The measurement of pressures transmitted through granular materials has always been difficult, due primarily to the fact that the measuring instrument must be capable of indicating such pressures with only an infinitesimal movement of the instrument itself. The very slightest motion of the weighing face disturbs the soil so that the pressure originally existing may be changed considerably at the instant the measurement is made. Any instrument to be successful for this purpose must be capable of measuring pressures with practically no motion, and the soil-pressure cell mentioned by Mr. Hazen fulfills this requirement.

The principle of its action may best be understood by referring to the cross-section shown in Fig. 16. The underlying idea of this device is to equilibrate the external earth pressure on the weighing face by means of internal air pressure, detecting the instant at which equilibration takes place through the breaking of an electrical circuit within the instrument, and at that instant noting the air pressure required to overbalance the earth pressure.

* Washington, D. C.

Mr. Goldbeck.

Referring to Fig. 16, the soil-pressure cell is seen to consist of a cast-iron base, *C*, 5½ in. in diameter, in the center of which is cemented a steel button, *E*. Bakelite cement is used for this purpose for the double reason that it is an insulator as well as a cement. A thin brass diaphragm, as thin and flexible as a sheet of paper, is stretched across the base, *C*, and is held in place by the malleable cast-iron ring, *D*. This diaphragm is clamped between two cast-iron disks, *A* and *B*, the


#10-32, steel, ½" long.
#12-24, steel, 1" long.
Material:
A, B, C—Cast Iron.
D—Malleable Cast Iron.
E—Steel.
Note: All interior surfaces to be coated thoroughly with vaseline.
Note: All joints withstanding air pressure to be treated with a special compound of rubber, vaseline and beeswax.
Note. Top to be covered with a brass diaphragm 0.002" thick, pressed over the sides and cemented, to sides only, with asphaltic cement. Brass top to be protected with covering of cheese-cloth cemented to top and sides with a blown oil asphalt.
PLAN
VERTICAL SECTION
Brass Diaphragm 0.002" Thick.
16 Copper Wire, Triple.
Braid
Weatherproof Ins.
⅛" Galvanized Iron Pipe.
Bakelite Cement No.5

DIAPHRAGM CELL FOR DETERMINING SOIL PRESSURE.

FIG. 16.

lower disk, *B*, bearing on the button, *E*. In order to protect the cell from moisture and to prevent the small annular space between Ring *D* and Disk *A* from becoming clogged with soil, the top of the cell is covered with a brass diaphragm like that between Disks *A* and *B*, and the entire instrument is coated with a layer of cheese-cloth cemented with a blown-oil asphalt. Blown-oil is used for this purpose because of its comparative unsusceptibility to temperature change. A galvanized-

iron pipe connects the pressure cell with a small tank of compressed air which is placed in a portable indicating box containing pressure gauges for reading the air pressure within the cell. An insulated electric wire is led through this pipe to the button, *E*, and the pipe itself furnishes the return circuit. A small dry cell and a miniature electric light are the only other electrical equipment required. The indicating box used with this pressure cell is shown in Fig. 17. Proper control valves are also placed in the air line in order to control very delicately the flow of air from the tank into the pressure cell. Mr. Goldbeck.

Let it be supposed that this instrument is to be used for measuring the pressures back of a retaining wall. The cells are first mounted securely against the wall with their weighing face placed toward the earth fill. It is necessary, of course, to mount the cells before the fill is in place. The pipe connections are led up to an accessible position, and if the cells are to remain in place and the readings continued over a long period, it is desirable to use a double pipe connection, as shown in Fig. 18. Should any water collect in the pipes through condensation or leakage, this fact can be determined very readily merely by allowing the air to flow into one side of the pipe line, the other end being open. If water is present, pressure will be indicated in the air line due to the head of water. The mere fact that water is present, however, does not necessarily spoil the readings of the apparatus, for by the use of a double pipe line, the operator can cause the water to mount into one side of the line while he is taking his readings, which are thus not impaired by its presence.

The operation in taking readings is to open the control valve on the pressure tank, thus allowing the air to flow very slowly into the pressure cell. As long as the cast-iron disk, *B*, in Fig. 16 is in contact with the steel button, *E*, this fact will be indicated by the burning of a small electric light in the indicating box. At the instant, however, that the air pressure within the cell slightly exceeds the soil pressure acting on the outside of the cell, an exceedingly small movement of the plate, *B*, takes place, and it is raised out of contact with the steel button, *E*, the electric light is extinguished, and the operator immediately reads the pressure on one of the test gauges in the indicating box. This records the air pressure within the cell, which necessarily equals the earth pressure acting on the cell. At the same instant the operator releases the air pressure within the cell, for at no time should he allow the air pressure to exceed the soil pressure by more than 0.1 lb. per sq. in., since by so doing the accuracy of future readings would be impaired.

The question of the accuracy of this instrument has been raised by Mr. Hazen, and it may be stated that it was several years before means could be devised for proving the accuracy of the results obtained where measurements of pressure of granular materials were made,

Mr. Goldbeck.

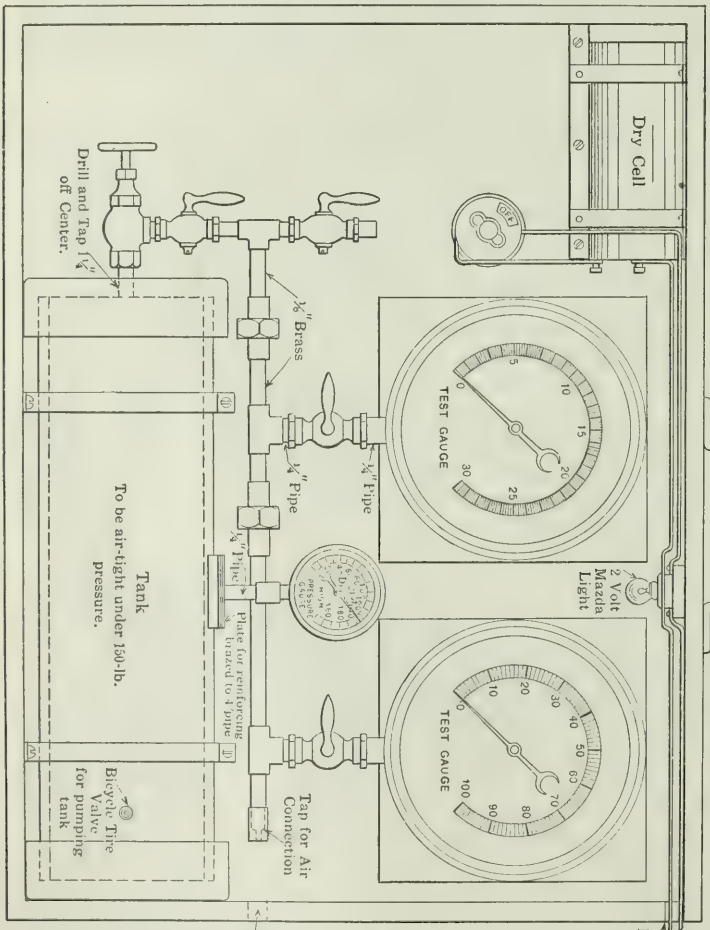
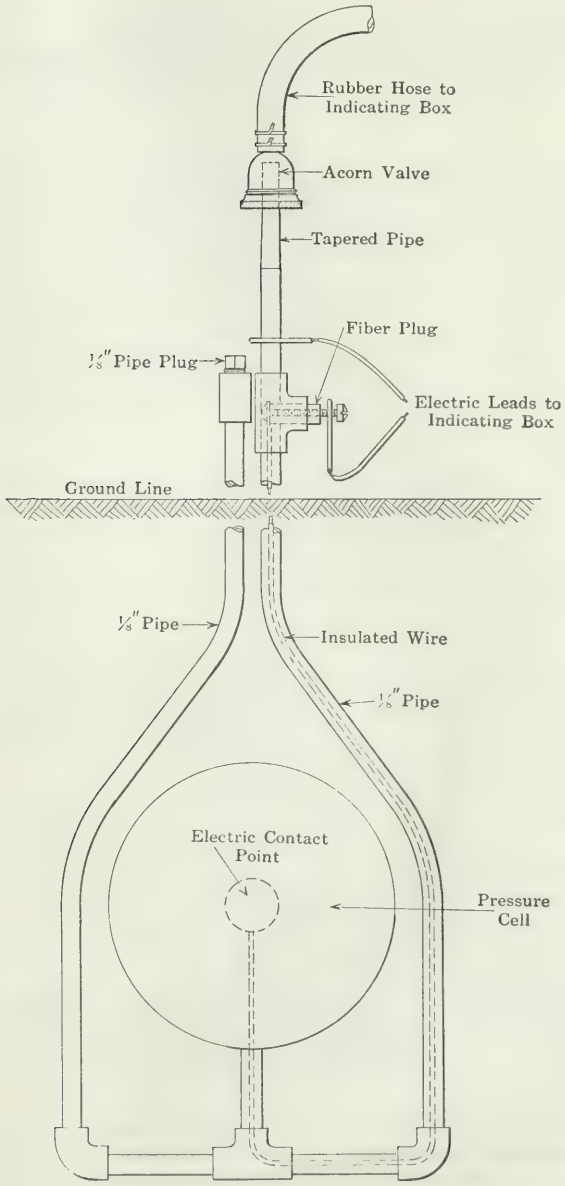

MOUNTING OF SOIL PRESSURE INDICATING APPARATUS.
FOR USE WITH DIAPHRAGM CELLS.
Handle to be placed over center of gravity of box and apparatus.
Dry Cell
2 Volt Mazda Light
TEST GAUGE
TEST GAUGE
Hole for Electric Leads
Hole for Air Hose
Drill and Tap 1/4" off Center.
3/8" Brass
1/4" Pipe
1/4" Pipe
1/4" Pipe
Plate for reinforcing brazed to 1/4 pipe
Tap for Air Connection
Tank
To be air-tight under 150-lb. pressure.
Bicycle Tire Valve for pumping tank

FIG. 17.

Mr. Goldbeck.



PIPE AND ELECTRICAL CONNECTIONS OF SOIL PRESSURE INDICATING CELL.

FIG. 18.

Mr. Goldbeck.

although it was a comparatively easy matter to prove the accuracy of the instrument when used in liquids or semi-liquids.

The method pursued in checking these results for granular materials consisted in measuring the movement of the disk, *B*, during the

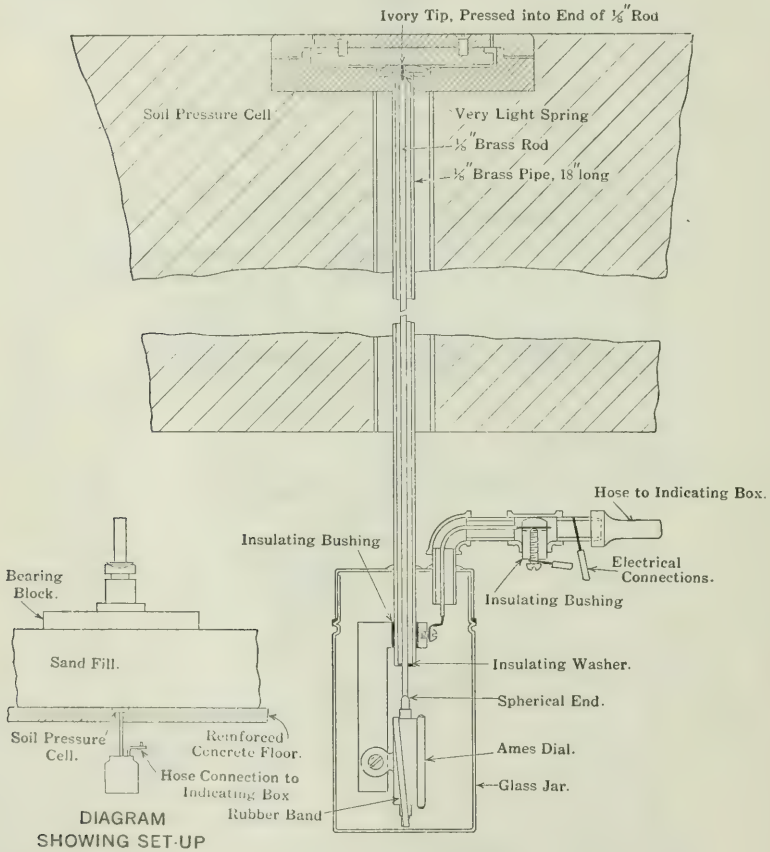


FIG. 19.

breaking of the electrical contact. For this purpose a special set-up of the instrument was used, as shown by Fig. 19. The cell was placed flush with the surface of a reinforced concrete floor and 3 ft. of sand was placed over the cell. In place of the wire ordinarily used in the pipe line leading to the cell, a brass rod having an ivory tip bearing

Mr. Goldbeck.

CURVES SHOWING MOVEMENT OF UPPER DISK OF SOIL PRESSURE CELL AFTER BREAKING ELECTRICAL CONTACT—30 IN. MOIST SAND FILL.

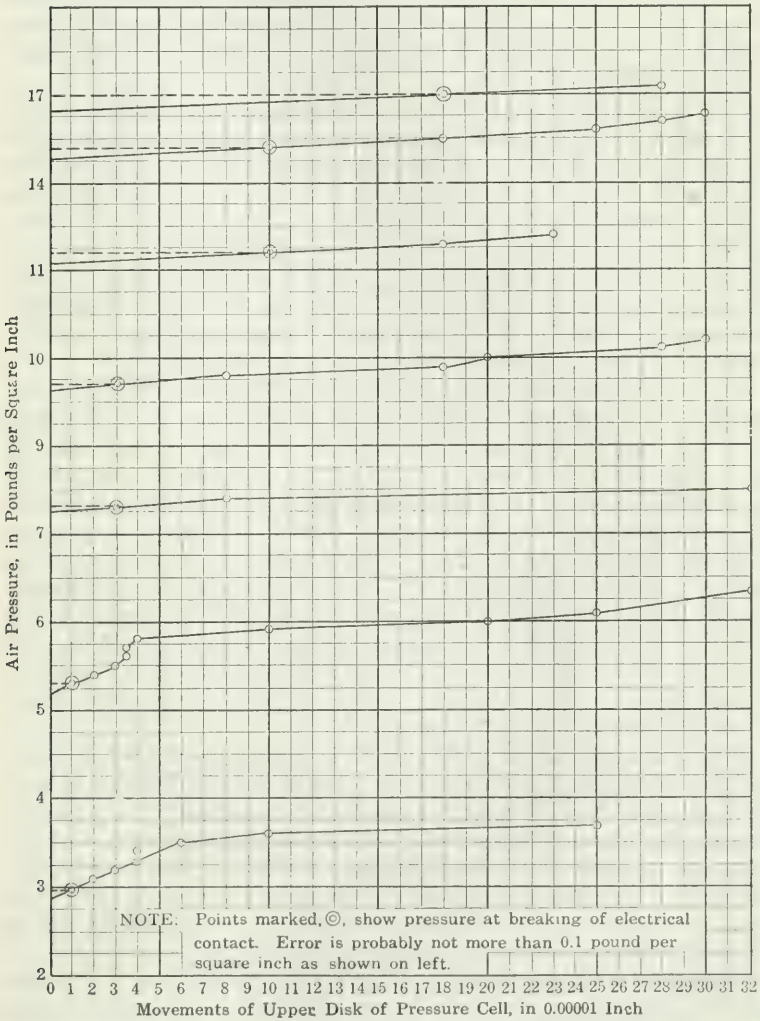

Air Pressure, in Pounds per Square Inch
17
14
11
10
9
7
6
5
4
3
2
NOTE. Points marked, ©, show pressure at breaking of electrical contact. Error is probably not more than 0.1 pound per square inch as shown on left.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32
Movements of Upper Disk of Pressure Cell, in 0.00001 Inch

FIG. 20.

Mr. Goldbeck.

on the movable disk of the cell was used. The other end of this rod bore on the plunger of an Ames dial reading movements of 0.0001 in. Concentrated loads were placed on a bearing block resting on the sand fill over the cell and thus definite unit pressures were transmitted to the cell. Air was introduced and simultaneous readings of the air pressure in the cell and movements of the movable diaphragm were recorded at the breaking of the electrical contact, and also after contact was broken.

The curves on Fig. 20 show these simultaneous readings. The double circle shows how much movement took place in the instrument up to the point of breaking the electrical contact, and the circles to the right of the double circle indicate the movements after the breaking of such contact. Before the electrical contact is broken the soil pressure is counterbalanced by the internal air pressure plus whatever pressure exists on the steel button, *E*. After contact is broken within the cell, the air pressure alone counterbalances the soil pressure and, therefore, it is known definitely how much increase in internal pressure is required to produce definite movements of the movable disk.

It will be noted that the movement required to break the electrical contact for external pressures up to about 15 lb. per sq. in. is less than 0.0001 in. and, for low pressures, only 0.00001 in. It will also be noted from the curves that, in general, an increase in pressure of less than 0.1 lb. per sq. in. is required in order to move the disk against the earth pressure 0.0001 in., or less. The small increase in pressure necessary to produce the exceedingly small movement of the diaphragm required to break the electrical contact is the error of the instrument, and this is not more than 0.1 lb. per sq. in., since, in general, a movement of less than 0.0001 in. is required to break the electrical contact. By virtue of the exceedingly small movement required to break the contact, the pressures in granular materials may be accurately determined with this type of instrument.

Numerous applications of this device have been made for measuring pressures transmitted through earth. Thus, the distribution of concentrated loads through sand fills has been determined.* In addition, pressures have been measured back of several retaining walls. At the suggestion of Charles H. Paul, M. Am. Soc. C. E., Assistant Chief Engineer of the Miami Conservancy District, the Bureau of Public Roads conducted an extensive series of tests in an hydraulic fill, in co-operation with the Miami Conservancy District, the test being made in a stand-pipe 41 ft. in height and 30 in. in diameter.† Typical curves resulting from this test correspond very well with those obtained by the Miami Conservancy District in pressure measurements

* "The Distribution of Pressures Through Earth Fills", *Proceedings*, Am. Soc. for Testing Materials, Vol. XVII (1917).

† "Tests to Determine Pressures Due to Hydraulic Fills", *Engineering News-Record*, April 18th, 1918, p. 758.

made with the same apparatus in hydraulic-fill dams.* The Bureau of Public Roads has also made measurements of pressures of concrete against forms with this instrument,† and has utilized it for making measurements of pressure transmitted to the sub-grades of roads.‡ Mr. Goldbeck.

As a result of these tests it was demonstrated that pressure cells of this character may be used successfully for measuring pressures under a variety of circumstances, even in hydraulic fills, for periods up to two years and perhaps longer. The hydraulic-fill pressure measurements demonstrated that material such as is used in the core of an hydraulic-fill dam acts as a perfect fluid during the early stages of settlement, but that it loses its fluid qualities as consolidation proceeds.

As a result of experience with this instrument, the writer believes it to give thoroughly reliable results, not only in liquids and semi-liquids, but even in granular materials such as densely compacted sand. It has also been demonstrated that the instrument is sufficiently rugged to withstand field conditions, and it might very readily be applied in obtaining useful data on many types of construction.

GEORGE L. DILLMAN,§ M. AM. SOC. C. E. (by letter).—This paper is a distinct addition to the literature on earth dams. It is also timely. Mr. Dillman.

Before entering upon the discussion, the writer would call Mr. Hazen's attention to one misstatement of fact. Messrs. Schuyler and Howells were not, as he states, pioneers in hydraulic-dam construction. The Chabot or San Leandro Dam was built before they were ever heard of, was largely hydraulicked, was for years the highest earth dam in the world, was never a failure or a partial failure, and suffered no accident during construction.

The analyses of earth dams and slides are identical. Slides have occurred during the construction of many earth dams, due to badly drained core material. The writer refers to the Pilarcitos, Necaxa, and Calaveras Dams. Drainage will avoid such accidents, just as it will cure any slide. The hydraulic-fill type makes a dam of each toe, the pressure coming from the core. When the pressure is too great, failure occurs, not failure of the dam as a whole, but failure of a portion of the toe.

The film of water behind any dam, behind any toe, or behind any slide, has only to be thick enough to be continuous in order to transmit the full pressure of its head under, into, and against the resistant

* "Study of Pressures in Hydraulic Dam Cores", *Engineering News-Record*, December 25th, 1919.

† *Proceedings*, Am. Concrete Inst., 1920, or *Engineering News-Record*.

‡ *Public Roads*, April, 1919.

§ San Francisco, Cal.

Mr. Dillman.

material. It acts like myriads of jack-screws to displace material, and it is often effective, as is proven by the movement of the material.

Drainage may be had in various ways. Mr. Hazen's plan for discarding the fine material is one of them; the coarse material drains itself. Building slowly is another, and building the cores narrow is still another. Building the toes to stand the pressure is effective, but is wasteful of material. The writer would suggest strings of tiles. They are cheaper and fully as effective as any other method; the expense of segregation and discard of fine material suggested by Mr. Hazen is avoided and, on the other hand, excess of material in the toes.

The object in an hydraulic-fill dam (or slide) is to break up the continuity of the film of water, thus killing the head and cause of the trouble. Tile drains are exceedingly cheap, and, properly placed, they will not only stop such accidents, but increase the factor of safety in the completed structure by making the "water-tight surface" or "most nearly water-tight surface", farther up stream than it would be without them.

Safety in a dam is not always reached by tightness. Making it tight in the wrong place is often more dangerous than leaks. In 1916, the writer planned, on a pervious foundation, with pervious material, a dam designed to leak. It does leak, some 160 sec-ft., with 50 ft. of water in the reservoir. There was some active objection, first, because it leaked and, therefore, was dangerous, and afterward, that it was too tight and depleted the ground-water below; both these objections came from the same source, which only goes to show that critics, like the poor, are always with us.

If it is ever desired to decrease the seepage, the writer will advise hydraulicking some fine material over the face of the dam. The dam referred to is the Mackay Dam, on Big Lost River, in Idaho. It is an interesting structure, built strictly in accord with the "great hydraulic principle" announced some 17 or 18 years ago, and some day the writer plans to write a description of it.

He can see no objection to fine material in a dam, if it is placed where it belongs; in fact, he rather favors it.

Mr. Muckleston.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—This interesting and timely paper calls attention to certain properties of the colloidal clays which appear to be responsible for the observed phenomena. Sands (which term includes all rock powder, no matter how fine) will settle in water in time, and form a stable sediment. The particles of clay in a colloidal condition or state may remain in suspension for an extraordinary time, measured by years in some cases, and form when they do finally settle, not a stable sediment, but rather

* Lethbridge, Alberta, Canada.

a viscous fluid or jelly which retains water to the extent of 60 to 70% or more of the whole, and will not consolidate further no matter how much time is allowed. Extreme pressure, with the addition of some heat, may convert this mass into shale or slate by expelling the water, but neither of these agencies can be active in the case of hydraulic fills, and, consequently, it is doubtful whether such fills ever do become even reasonably dry. Mr. Muckleston.

The writer has met with this phenomenon on two occasions. On one such occasion a shallow lake formed the most convenient supply of water for camp use. The lake was only about 1 ft. deep, with a blue clay bottom, and the water looked much like good gray paint. Filtering was tried, but without success as no water got through the filter. Sedimentation gave no better results, as the only effect of a week's trial was a thin ring film round the top of the barrel, due to evaporation. A 1-pint sample stood in a glass bottle for more than a year and seemed to be just as turbid as when freshly poured in, although the contents at about 20% of the height of the bottle appeared to be a darker color than the remainder. The water was then allowed to evaporate and left a deposit, apparently about 10% of the original volume, of a light gray powder, much resembling talcum powder, in everything except color, and having no gritty feeling between the teeth.

This clay is the result of hydraulic selection. The lake never has more than about a foot of water and, usually, in four years out of six, none at all except in early spring. It has no outlet. In consequence, the finest particles of soil from the surrounding country have been washed down, and as the water, when there is any, is continually stirred up by wind, the finest particles have been gradually brought to the surface with the colloidal particles on top.

The other occurrence was an attempt to back-fill a retaining wall by hydraulic means. The material available was a sandy clay soil containing small pea-size pebbles, but there must have been an undue proportion of the same kind of clay, because the fill never did consolidate and more than a year afterward was removed by boring holes in the face of the wall and allowing it to flow out. The holes were 1 in. in diameter and the wall 30 in. thick, yet the material poured out much like thick molasses.

This colloidal clay may be thrown down by some of the common mineral salts and in this condition looks like boiled starch with or without some color, and when the water is expelled resembles glue in appearance more than the ordinary conception of clay. It has an extraordinary affinity for water, which it absorbs with great energy, rapidly swelling to its former starchy consistency. If given a less quantity, it is much like pitch—tenacious, sticky, and as hard to handle.

Mr. Muckleston.

The alkalies, such as carbonates of sodium and potassium, act in the reverse way to the acids or neutral salts, resulting in a hard impervious substance which is the base of the soils generally known as hardpan or the like.

Mr. Dunham.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—If it is true that "all earth dams leak", some water must have found its way from the construction pool through the toe on the down-stream side of the Calaveras Dam. No movement or bulging of the earth occurred there to show instability. The factor against such movement has been calculated. Still, it would be of interest to learn what the leakage there amounted to, how general it was, and the elevation at which it occurred or was most noticeable. The two sides were quite similar except that the down-stream side had the more abrupt slope, as the description indicates, but in one respect there was a marked difference. On the down-stream side or toe all leakage was free to escape. On the up-stream toe there was, first, a concrete facing and then the water in the reservoir to hold back any leakage and insure more complete saturation of the material filled in by dump-car or by hydraulic methods.

A cross-section of the dam as it was at the date of the failure showing the height of the concrete facing, the water level in the reservoir, and the known-to-be lowest elevation of very soft or plastic material in the core would be of great interest. Of almost equal value would be a knowledge of the rate of construction of the concrete facing and of the rate of increase in depth of water in the reservoir. Together with these items, the known facts about the quantity or extent of the leakage through the down-stream toe should be recorded, as well as the known leakage on the up-stream side. Without all the data, it would seem to be impossible to make reliable calculations as to stability or devise any factor of safety against sliding movements. So far as the paper shows, the items mentioned were not considered together in their relation to the failure.

As a related feature: If it was noticed at any time during the work that the shore lines of the construction pool were irregular and that the bays extended toward the toes on either side, the elevation of such bays and the distance from the center line of the dam to the shore line should be of interest now, especially if it appeared that the outward bulging on the up-stream slope occurred opposite a bay-head where much fine material had been deposited.

The sliding factor or coefficient of friction may have been assumed for movement along a horizontal plane, but if the up-stream toe had become saturated to the level of the water in the reservoir, while the core and other material above were saturated to an unknown depth,

* New York City.

there would be reason to think that the sliding surface was inclined from the horizontal or toward the base of the outlet tower. How, then, could the real sliding factor be obtained unless that angle of inclination could first be determined? The water in the reservoir would tend to lessen the weight of some of the material in the toe depending, of course, on the extent of the saturation. Mr. Dunham.

Recalling the favorable difference in slope of the two retaining halves or toes of the dam and their great similarity otherwise, there would be some reason to infer that anxiety to complete the concrete facing and to store water led to the Calaveras disaster. If so, there may be a valuable object lesson in the occurrence, in addition to the lessons derived from calculated factors.

Some surprise was caused by the firmness of the core material after the failure. Energy to the extent of many million-foot tons had been expended in that vicinity and in a very short interval of time. A part of that energy may have caused displacement or removal of moisture from the core material. If a large mass of wet concrete falls a considerable distance and stops abruptly, a portion of it will be found to be quite hard and to hold comparatively little water. Had the entire clay core been prepared in some convenient niche in the mountain, several hundred feet above the dam site, and then shot down in 3 or 4 min., the water-tightness would no doubt have been satisfactory. The writer claims no originality for this suggestion. It would be only a feeble imitation of Nature's way in formative processes involving motion and pressure.

William H. Burr,* M. Am. Soc. C. E. (by letter).—The hydraulic method for building earth dams has experienced the usual vicissitudes of a comparatively new procedure. The author of this interesting paper has indicated that the method of building an earth dam by hydraulicking the material into place is scarcely more than from 20 to 25 years old. He further observes that the applications of the method have been marked by failures of magnitude, which are due to defective plans of procedure, and not the least prominent among such failures is that of the original Calaveras Dam itself. His paper presents the physical conditions prior to the failure of the dam, which constitute essentially a demonstration of the reasons for the failure. Mr. Burr.

In order that the fine clayey material of the core deposited by sedimentation may constitute a water-tight mass of material, it must, clearly, be divided into minute particles or grains, and apparently it has been assumed in the past that such material cannot be divided too minutely. At first sight this seems to be plausible enough, but the physical analyses of the core material at Calaveras have shown what might be expected, that is, that the finely divided material with

* New York City.

Mr. Burr. an effective size of 0.001 to 0.003 mm., in which the voids may be from 35 to 40% of the total dry volume, has a capillarity of such intensity and affords such great resistance to the flow of water through it, that it is practically impossible to consolidate it by drainage within a reasonable period of time, if at all. It is Mr. Hazen's opinion that no material of an effective size of less than about 0.01 mm. should be used, that being about the smallest grain that will permit suitable drainage, and he is undoubtedly correct in taking that position.

His conclusions are based on extended investigations of the physical character of the material found at Calaveras after the failure, including not only the voids of the core material but the quantity of water held in them, which had not drained out apparently to any material degree, even after lying in the partly completed core volume for more than a year. The experience gained through the failure of this dam and the demonstrative character of the investigations made prove conclusively what has happened in prior and similar failures. The excessively fine material, separated by the hydraulic process and carried into the pond between the two toes of coarse material, refused to give up its water and consolidate. There is little or no reason to suppose that it would drain within a reasonable period of time, even though the supersaturation is as much as 20 per cent. These observations are probably true, irrespective of the character of the core material, provided the effective sizes of the minutely divided particles or grains are no larger than those found at Calaveras. The intense capillarity existing under such circumstances produces a semi-liquid, the intensity of pressure of which may be measured by the head or distance below the free surface.

The possible results attending such a condition of more or less liquidity are generally recognized, but there are some consequences which apparently have not been fully appreciated, even in the Calaveras case. This supersaturated, more or less liquid, mud is practically the same in kind as the vast mud deposits—some subaqueous and others not—which are found in many places. Piles driven into such material are said to carry their loads by "mud flotation". That material never consolidates, nor is there reason to believe confidently that finely divided material in similar condition will drain or consolidate anywhere, even under conditions like those found in an hydraulic-fill dam.

In the Calaveras Dam, therefore, and in other similar hydraulic-fill dams, the core was at least a semi-liquid mass exerting essentially liquid pressure against the coarse material on either side of it, and this observation is not adversely affected by the fact that the demarcation between the coarse and the fine material is not well defined. The author shows all this in a clear and conclusive manner. The large

horizontal forces thus caused to act against the coarse material of the two toes, however, is not all of the influence of this semi-liquid mass in producing displacement and failure of the two toes. The pool was much wider at the bottom than higher up toward the top of the dam. This produced an overhang of coarse consolidated material which in the Calaveras Dam was considerable. The result was a vertical upward pressure against this overhang, varying with the depth below the surface of the pool. This pressure was great enough to be of material effect in aiding the original failure. As it would tend to relieve the pressure on the bottom of the coarse material near the toe, it should be taken into account in considering the coefficient of friction on that bottom. Mr. Burr.

Again, the larger part of the coarse material of the up-stream face was submerged by the water in the reservoir when the failure occurred. All materials so submerged would of course be diminished in apparent weight by the quantity of water displaced. Inasmuch as all the material of the up-stream toe varies in size from gravel to fragments of rock of considerable size, the water would percolate freely through a large amount of the mass, thus relieving the load on the base of the latter. This, also, should be taken into account in the endeavor to find a coefficient of friction, although the manner in which the mass of coarse and fine granular material is pushed along in such violent displacement, induces a condition quite different from that which exists when a heavy continuous body is, in the strict sense of the word, pushed over a continuous surface in the manner of a true slide. In fact, the motion of such a heterogeneous mass of granular material (including even rock fragments) will partake at some points and, perhaps, at a considerable number of points, at least partly, of the nature of confused rolling and sliding, the resistance to which would be much less than that offered by a more or less rigid mass sliding on an irregular rough surface.

Furthermore, the upward pressure of the semi-liquid against the overhang of the coarse material would tend to produce, and, to some extent, undoubtedly would produce, horizontal or inclined openings in that material closely adjacent to the pool and near its bottom, into which some fine material would find its way and act as a lubricant. Whether any quicksand condition would intervene before the motion of the coarse material begins may well be doubted. The rather indefinite material called quicksand does not take on its "quick" character until the water producing supersaturation takes on motion. It scarcely seems to be necessary to imagine that any such condition acted to aid failure prior to the motion of the coarse material of the toes. The lateral and vertical pressure of the supersaturated core material against the up-stream toe, together with the buoyant relief given to

Mr. Burr. the volume of the coarse material of the up-stream toe due to submergence, is sufficient to account for all that took place.

A review of the literature of this class of engineering work does not disclose a thorough and systematic examination and control of the core material which has hitherto been placed in hydraulic-fill dams. Various rather crude tests of the consistency or degree of consolidation of this material have been made from time to time during the progress of the work, which are well enough as far as they go, but they have fallen seriously short of the necessities of the case by their failure to give quantitative results. No hydraulic-fill dam can be considered as being safely put in place without careful investigations as to the size and supersaturation of the core material at practically all points and at all stages of the work. It is imperatively essential that the character and deposition of that material be completely controlled. A semi-liquid mud will resist a degree of penetration of a heavy object which may be and usually is highly deceptive as a test of consolidation, as pile-driving in such materials constantly demonstrates.

The procedures at Calaveras and San Pablo, after the failure of the former, as set forth by the author constitute probably the first systematic controlling treatment of core material of this kind. The Calaveras failure, therefore, like many other failures, has proven to be of great value to this important field of earth dam construction. It has shown the necessity of determining and limiting the fineness of core material, the amount of its voids, and degree of supersaturation, to the end that its physical characteristics and the process of consolidation may be completely controlled at all stages of the work.

It appears to the writer that with work performed under effective control, it would be desirable to design hydraulic-fill dams with much less volume of core material than has been used heretofore, that is, with a much less width of core. At any rate, material of excessive fineness, as Mr. Hazen indicates, may be profitably wasted.

Mr. O'Shaughnessy. M. M. O'Shaughnessy,* M. Am. Soc. C. E. (by letter).—This paper presents an interesting résumé of the hydraulic method of constructing dams. The author has added little that is new on this subject beyond that already furnished by J. M. Howells, M. Am. Soc. C. E., and the late James D. Schuyler, M. Am. Soc. C. E., who made successful applications of this process.

The character of the adjacent materials to be incorporated into the structure dominates the type of dam to be constructed in any particular locality. In Southern California, where Mr. Howells has made successful applications of the hydraulic-fill process, the washed detritus is of a gravelly nature which separates mechanically on deposition in

* San Francisco, Cal.

Mr. O'Shaughnessy.

the dam in the manner prescribed by the engineers, the coarser materials going to the extreme up-stream and down-stream portions and the finer materials toward the core. The porosity of the coarse materials permits drainage of the mass, which leaves the structure fairly safe if work is not prosecuted too rapidly.

An intimate knowledge of the geological and mineral characteristics of the neighboring locality, obtained by study and exploration, is a necessary qualification of engineering supervision, in order to build a hydraulic-fill dam which will be safe.

The terrain of Calaveras Valley, where the Calaveras Dam was built, is of a decidedly clayey nature, as indicated by the material deposited in the completed portion of the dam; and the trouble with that structure resulted from the excessive quantity of clay and the fact that the materials deposited by the hydraulic process, on both the up-stream and down-stream slopes, constituted an admirable core material for an earth dam, if the necessary gravel was added, and this very quality prevented the proper drainage of the structure.

When this work was started in 1913, there was no material in the vicinity of Calaveras Valley suitable for hydraulic-fill dam construction. The gravels and sands along the river-bed carried a large content of clay, which was deposited in the foundation portions of the dam, and as the down-stream and up-stream toes are always kept higher than the central pool, they constituted barriers to prevent the drainage of the pool. The original plans for the dam called for a porous material in both the up-stream and down-stream slopes, and it is very clear, therefore, that the material distribution called for by the plans was not followed in the construction of the dam, due to the excess of clayey contents encountered.

No difficulties of this kind have been experienced in the valleys of Southern California, San Fernando, and other places, where the material washed off the mountains is a detritus from crystalline rocks and includes a very small percentage of clay.

The dominant claim of advocates of this type of dam construction is the cheapness of the hydraulic-fill process, but the writer is advised by G. A. M. Elliott, M. Am. Soc. C. E., that, in the case of the Calaveras Dam, this result was not in accordance with predictions. The initial estimate was about 20 cents per cu. yd., while the actual cost was more than 45 cents, and this was in a time of cheap labor. On taking into account the cost of the pipe and the wear on it, the cost of the pumps and their depreciation, and other expenses, it seems to the writer that the steam shovel and good railway equipment would have been much more economical.

It is a wise precaution to put a rock fill both on the up-stream and down-stream toes of all earthen dams. This construction was used by

Mr. O'Shaughnessy.

the writer about 10 years ago on the Throttle Dam, about 70 ft. high, 18 miles southeast of Raton, N. Mex. This dam had a conventional clay puddle core, 20 ft. thick at the bottom and 8 ft. thick at the top, which was built simultaneously with the rolled earth layers about 1 ft. thick, and well watered, on each side of the core. All stone found in the excavation was placed in the down-stream portion of the dam. The puddle core reached a cut-off wall of concrete, carried from bed-rock, at a depth of about 20 ft. into the structure. The down-stream portion of this dam was always kept higher than the up-stream portion, and notwithstanding the fact that floods as great as 8 000 sec-ft. were anticipated and some damage was expected during the construction (of which the owners were advised), this whole work was completed in a satisfactory manner, at a relatively moderate price, and is absolutely tight and safe, with two large spillways cut into the solid rock at each end of the dam.

On behalf of the City and County of San Francisco, which has now under consideration the purchase of the property, the writer has modified the plans prepared by Mr. Hazen, for the completion of the Calaveras Dam, lowering its height to a crest elevation of 775 ft., with the spillway level at an elevation of 753 ft. He has also provided for a rock toe on the up-stream side, and for bringing the construction up in layers, with heavy rolling and watering, in the conventional manner of constructing earthen dams.

The writer disapproved the suggestion of a dam of the Calaveras hydraulic-fill type at the site of the Alpine Dam in Marin County, California, and its adoption by the Directors of the Marin Municipal Water District would inevitably have resulted in a disaster similar to that which took place at Calaveras. A sane, gravity, arched, concrete dam on excellent bed-rock has now been completed at this site and provision made for raising it at a future date, at one-half the ultimate cost of a hydraulic-fill dam.

The true principles controlling dam design and construction have been clearly elucidated by George L. Dillman, M. Am. Soc. C. E., in his discussion* of the Morena Rock Fill Dam. If those principles are adhered to, either for a masonry, earth, or rock fill dam, there will be no danger of any disaster taking place on the completion of the work.

Mr. Morgan.

ARTHUR E. MORGAN,† M. AM. SOC. C. E. (by letter).—So much has yet to be learned about the principles and facts of hydraulic-fill dam construction that every item of added information is pertinent and valuable. Even expressions of opinion with which one cannot agree are of value when they suggest new points of view or methods of inquiry.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 52.

† Dayton, Ohio.

Mr. Hazen's opening statement is: "The hydraulic method of dam construction has great advantages, but too many dams built by it have failed." The searches of the engineers of the Miami Conservancy District have developed only two cases of failure of hydraulic-fill dams, namely, the Necaxa Dam and the Calaveras Dam. In case of other failures of so-called hydraulic-fill dams, it was found that this method of construction was not used. In some cases, efforts have been made to construct embankments by sluicing in materials without any attention to their segregation in the dam, and without the proper use of a central pool. Mr. Morgan.

In the case of the Necaxa Dam, conversations with persons employed there have led the writer to the opinion that the cause of this failure was not so difficult to determine as has been indicated by some of the written reports that have appeared, but rather was due to the failure to observe some of the elementary principles of safe construction. Among other conditions which have been described to the writer as contributing to the failure was that of the up-stream toe. He has been informed that an accumulation of muck and unstable material excavated from the site of the dam was allowed to remain within the cross-section on the up-stream side, and that it was at the location of this pile of débris and waste material that the failure occurred. If this information is correct, it would leave the Calaveras Dam as the single instance of the failure of a hydraulic-fill dam built under careful supervision. However, one failure is one too many.

Mr. Hazen's wide experience with sand filters has drawn his attention pointedly to drainage as one of the methods of consolidation of the core of the hydraulic-fill dam. The writer believes, however, that he is mistaken in considering drainage to be the only, or even the most important, factor in consolidation. About fifteen years ago, in the process of reclaiming wet lands, the writer had occasion to drain thirty or forty shallow lakes in Minnesota. These lakes were located in various parts of the State and under widely different soil conditions. Observation there indicated that the fineness of the material which, according to the author, determines the rapidity of drainage and consolidation, did not appear to be the most prominent factor.

In the western part of Minnesota the glacial drift forming the lake beds is composed of powdered limestone and shale in the form of a very fine blue or yellow clay. In some cases, these clay deposits had become so thoroughly compacted under water that a few weeks after the surface water had been drained away, a road roller could be driven over the surface. In the central part of the State some of the lake beds consisted of a mixture of sand, clay, and organic matter. Here, it was frequently possible to push a wooden pole 25 or 30 ft. into the soft mud in the lake bottoms. In the eastern part of the State, the writer has encountered deep deposits of powdered mica of the glacial period,

Mr. Morgan.

which were so unstable as to flow like water. These materials had remained in place through many centuries, and the powdered limestone deposited as erosion from the surrounding hills became almost as firm as hardpan. Between this and other material which retained almost the consistency of a fluid, nearly every degree of consolidation was encountered. The writer is of the impression that the physical and chemical composition of core material and the shape and specific gravity of the particles have much to do with its rate and degree of consolidation.

The consolidation of core material takes place to some extent by the drainage laterally and downward of the enclosed water, as the author suggests, but to a much greater degree by displacement of the water upward, due to vertical pressures and to the settling of the fine material into the voids. The author mentions this upward movement of the water, but his entire argument thereafter seems to repudiate or ignore it.

In the construction of the dams of the Miami Conservancy District, consolidation of the core material actually is taking place at a fairly rapid rate. It is doubtful whether more than a very small part of this consolidation results from drainage, as defined by Mr. Hazen. A clear distinction should be made between the compacting of material by drainage and that by consolidation through settling under water. In the construction of hydraulic-fill dams, engineers are but little concerned with the first method of compacting and very greatly concerned with the second method. As the author's paper is largely taken up with a discussion of methods for insuring drainage by the lateral removal of water, the writer doubts whether it is as applicable to the solidification of the cores of hydraulic-fill dams as to the operation of sand filters.

Recently, at the Lockington Dam of the Miami Conservancy District, the hydraulic fill was interrupted and the surface water drained from the pool. As the surface of the core became dry, there appeared upon it hundreds of small springs discharging water which was being displaced from the core. In each case, a cone was formed of clay carried out with the water. These cones would become about 1 in. high and about 1 ft. in diameter. At another hydraulic-fill dam where work was discontinued until the water in the central pool became clear, these little craters were everywhere evident on the surface of the core under the water, with water pouring out of them like small springs. The confined water evidently forms vertical channels, and through these channels, rather than through the small interstices between the clay particles, the water escapes from the core. The character of these vertical channels can be understood by observing the markings which sometimes appear on the removal of forms from vertical concrete surfaces, especially where an excess of water has been used in mixing the concrete.

That similar vertical channels are formed throughout the mass of newly placed concrete where an excess of water is used, is evident from the appearance on the surface of similar craters from which water pours. Evidently, the size of cement particles does not determine the rate of the ejection of surplus water from the concrete. Mr. Morgan.

Concerning hydraulic-fill dam construction Mr. Hazen states: "It has two fundamental advantages: First, in sorting out the fine particles and placing them in the center of the dam, thus insuring complete water-tightness; and second, in the use of power in place of muscle to the greatest extent." There are other material advantages of this type of construction. Work on a dry, rolled-fill dam commonly is interrupted during rainy weather. Work on a hydraulic-fill dam can continue almost regardless of rain and the condition of the surface. Where rapid progress or haste in making a closure is essential, continuous operation may become an important factor. A water-tight core is highly desirable in a dam, but so also are pervious toes. An ideal hydraulic-fill dam section is one in which the central portion is entirely impervious, whereas the outside portions are so porous that any leakage which may occur, will appear at the base of the dam where it can do no harm, rather than part way up the slope where it may cause a slip.

The author also states: "In general, all the material placed in hydraulic-fill dams may be divided into two classes, namely, toe material and core material." There are three fairly distinct classes of materials in a hydraulic-fill dam: First, the toe material which consists of coarse gravel and sand not readily transported by water; second, the core material consisting of extremely fine particles which remains in suspension, generally for days at a time, falling to the bottom only very slowly; and third, the transition zone between the core material and the toe material, which consists of fine sand, silt, and other material readily moved by water, but which will settle several feet in the course of an hour or so. It is distinctly different from either the core material or the toe material.

Mr. Hazen states concerning core materials: "As a practical proposition, drainage is not possible with such fine materials. By that is meant such prompt and complete drainage as will result in solidification during the construction of the dam." Nevertheless, the writer's experience indicates that consolidation and solidification do take place at a fairly rapid rate. At the Taylorsville Dam, the hydraulic fill was interrupted and a portion of the water drained away. The consolidation was so rapid that whereas at the time of stopping work an iron ball would sink 8 or 10 ft. in the core, within a few months it was possible to walk anywhere over the core, and a horse might even be driven over it. This is true both of that part of the core which was exposed to the air, and of that part which was permanently under water. About 40 or 50% of this core material is finer than the mini-

Mr. Morgan.

mum recommended by the author. A detailed analysis of this core material is presented in the discussion by Charles H. Paul, M. Am. Soc. C. E. When an auger hole was bored into the core, water would rise in the hole and flow out over the surface, being displaced by the weight of the superimposed core material.

Mr. Hazen gives favorable consideration to reducing the thickness of the core until it has almost disappeared, in this way eliminating much of the finest material. If the border line between the core material and the coarser material could be depended on to continue without change during the construction, this method might be feasible, although unnecessarily expensive, as a rule, since it would require the wasting of fine materials. There is no assurance, however, that such uniformity of core margins can be secured. If perfect workmanship and perfect control could be had during construction, one might be assured of an absolutely uniform core margin. As a matter of fact, the margin between coarse and fine material shifts within considerable limits. In the Conservancy District dams, a width of about 25 ft. is maintained on each side of the core where it is assumed that the condition is uncertain. In some cases, a tongue of toe material will shoot out beyond the theoretical lines and tend to make a gravel bar across the dam. In other cases, the flow of water into the pool will result in a bay or depression, leaving the core projecting into the toe material. With the greatest possible care, this fluctuation between toe material and core material can scarcely be held within closer limits than a belt perhaps 15 ft. wide on each side of the pool.

There may be cases where the cross-section of a hydraulic-fill dam might be reduced if lateral pressure of the core material could be eliminated. As a rule, there are other controlling conditions, and a dam properly designed from other points of view will be stable as far as pressure of core materials is concerned. In the Miami Conservancy dams the width of the core is held about equal to the distance down from the top of the dam. As these dams have steeper slopes at the top than at the bottom, the proportion of the width occupied by the core will vary. That the Calaveras Dam had too wide a core there is no doubt, and a definite limit of width of core is essential. In the design of the Conservancy dams, the possibility of failure during construction, due to excess width of core, was a prime consideration.

The experience of the engineers of the Miami Conservancy District leads to the conclusion that there are so many factors entering into the stability of a hydraulic-fill dam that it is not safe to single out any one or two of them and to rely on the theoretical determinations of those factors. The final conclusions are as apt to be empirical as theoretical; or, rather, many problems present themselves for solution, which never would be brought to the attention by theoretical deductions, but only by observation of construction in progress.

Mr. Morgan.

As the construction of the dams of the Miami Conservancy District nears completion, it is the intention of the engineers to make a careful report of their opinions based on this experience. As this construction work is scarcely half completed, it will be a year or more before these opinions and conclusions will be sufficiently definite for presentation.

Mr. Parsons.

H. deB. Parsons,* M. Am. Soc. C. E.—The author is to be congratulated on giving such a clear description of a most interesting subject. A rock-fill dam is not exactly in the category of an hydraulic-fill dam, because the material is not washed in by hydraulic streams, although such dams are made tight by the seepage of water through the rock fill itself. The speaker designed a rock-fill dam which was built in 1902, in the Ramapo Hills, near Ramsey, N. J., for the late Mr. George Crocker, of California.

Mr. Crocker desired a dam in order to secure a water supply for his estate, and he had planned a gravity-section stone dam, the plans for which he sent to the speaker for criticism and advice. After an examination of the site, the conclusion was reached that it would be a good opportunity to construct a rock-fill dam. There was rock adjacent to the site, and the earth had a certain quantity of clay mixed with it. The selection of a rock-fill dam was based on the expectation that its construction would be much cheaper and more suitable for the site than the original plan for a cut-stone gravity dam.

This dam is not to be compared, as far as size is concerned, with those which the author has mentioned. Although much smaller, the principles involved are true for dams of larger proportions. The dam was 230 ft. long on the crest, 20 ft. high, and had a top width of 15 ft. It was built with stone roughly dumped in the center, while on the down-stream and on the up-stream faces the stones were placed by hand, in order to obtain some regularity. The slope on the down-stream face was 1 on 1½, and that on the up-stream face 1 on 1, but against the up-stream side an additional earth embankment was placed. The stone was laid roughly. The rocks were brought down from the hillside quarries, and the men simply dumped the core material in place, while carrying up the side faces a little ahead of the center core.

At the up-stream toe there was a cut-off, merely sheet-piling driven into the soil, with some parts ditched and puddled with clay. The pond above the dam covered between 11 and 12 acres. The site was prepared by removing the top surface soil so as to receive the stone which was brought down from both hillsides.

Two 36-in. pipes were built through the dam in order to control the water in the pond, and concrete was used around them to prevent

* New York City.

Mr. Parsons.

leakage of water and to give them strength. They were steel pipes, which, after a time, would rust out, and the concrete was intended to remain and take their place.

The dam had no spillway. It was not expected to be overtopped except at long intervals, and it was designed so that if it was overtopped, a flood would do no harm. The reason for paving the downstream side with large flat stones, especially picked out, was to prevent injury in case the dam should be overflowed.

The dam was finished in six months, having been started in May and finished in November, and its cost was $8 700, so that it is not difficult to figure the saving which this rock-fill construction made

This particular dam leaked during the process of construction, but at the end of six months, when it was finished, it was practically tight. Although it continued to leak, it was substantially tight thirty days after it was finished. Fine material had washed into the interstices of the rock that formed the core and had sealed the voids.

About twenty-five years ago, when the speaker was connected with the Nicaragua Canal Construction Company, the engineers proposed to construct the Ochoa Dam for the purpose of converting the Rio San Juan into a canal. This dam was designed to have an average height of 61 ft. above the river bed, a total length, including concrete abutments, of 1 900 ft., and a width at the base varying from 400 to 1 000 ft. It was proposed to construct the dam by depositing loose rocks and boulders which were expected to become water-tight by the lodgment of clay, sand, and drift. Although the dam received considerable discussion at the time, no question was raised as to its stability. There are locations where a rock-fill dam is more suitable than an hydraulic-fill dam.

Mr. Binckley.

George Sydney Binckley,* M. Am. Soc. C. E.—The speaker is especially interested in Mr. Hazen's description of the Calaveras Dam and the causes which contributed to its disastrous failure. In this connection there are several points of importance to which it would seem pertinent to call attention. In the first place, where heavy material is deposited in the outer slopes of the structure by running water, the material is not only free from lubricant or light fines, but is incomparably better packed than when it is dumped from wagons or cars. This fact, together with the automatic segregation of coarse from fine materials, which should be characteristic of a well built hydraulic-fill bank, is something that should by no means be lost sight of when such a bank is compared with one constructed as a dry rock fill.

As the speaker pointed out in an article† soon after the failure of the lower Otay Dam some years ago, a dry rock fill of unsegregated

* Los Angeles, Cal.

† *Engineering Record*, June 10th, 1916, p. 769.

material is, if overtopped, capable of becoming gorged with water and in this condition its resistance to movement is enormously decreased, the hydrostatic pressure within the mass tending to its disruption, and its effective weight being decreased by the weight of water displaced. This condition as a matter of fact is what caused the destruction of the Lower Otay Dam, as the best of evidence exists to prove that erosion by the overpour had little or nothing to do with it. This example is mentioned for the purpose of emphasizing the difference in the character of slopes constructed by the hydraulic-fill process, and that of the outer slopes of the Calaveras Dam. Mr. Hazen states that during the earlier stages of construction, the pond, and, therefore, the puddle deposited, extended at times over the greater part of the width of the dam. This fact alone would be quite sufficient to account for the failure, for under such conditions a stratum of very plastic clay could exist between a base of coarser material and a superimposed mass through which the clay could not be extruded. If there is no outlet for the extrusion of a fluid or plastic material, it will support very great unit pressures while, at the same time, retaining its fluid or plastic character. Hence, if at any plane in the slopes of the dam there existed any such stratum of plastic clay, cause for surprise would lie, not in the failure, but in the endurance, of the structure. Mr. Binckley.

Mr. Hazen mentions the possibility that under pressure the coefficient of friction might not be the same as under the conditions existing in the tests. It would seem to be quite safe to assume this possibility, for in the case of a material containing a more or less plastic clay in sufficient quantity to fill the voids, the unit resistance of the clay to change of form would be a constant, while the forces acting to produce movement along any given plane within the mass are variable. The conclusion seems inevitable that, under such conditions, the "coefficient of friction" (which in the case of the dam is actually a shear along the plane of greatest unit stress) would be higher where the forces in action are small, and rapidly decrease as such forces became greater in relation to the resistance of the plastic material to flowage or distortion.

The phenomenon of fluidity under motion of a material which is quite solid when it comes to rest, is mentioned by Mr. Hazen as having been observed in material which ran out from the Calaveras Dam. This is exactly the result to be expected if considered in the light of the foregoing remarks on the subject of the coefficient of friction. The same phenomenon was observed in the case of the Necaxa Dam. Great masses of material which flowed like water on a low gradient were found, when the motion ceased, to be solid, well compacted material containing relatively little water. More striking still are the great rock flows that may sometimes be seen in the West, a classic example being that of Turtle Mountain, in British Columbia, where an enormous

Mr. Binckley.

mass of rock slid off the top of the mountain, ran across a valley and well up the opposite slope before it came to rest—as solid a mass of rock and wreckage as if it had been built in place.

That part of the paper dealing with the Necaxa Dam has for the speaker a particular interest through his long association with the late James Dix Schuyler, M. Am. Soc. C. E., who designed this dam, and under whose direction it was built. In this connection, a little inside history relating to the conditions which preceded the accident to this structure may be of interest. Several months before the break in the up-stream slope occurred, Mr. Schuyler was showing the speaker the progress reports and cross-sections of the dam, and commenting on a curious condition which had developed—the rising of the core material at several points in the pond maintained above the core of the dam, to such an extent that islands were formed. This seemed somewhat of a puzzle, but after the speaker had suggested that it was caused by units of the heavy material of the slopes sinking into the core material and displacing it, he agreed that this was probably the explanation.

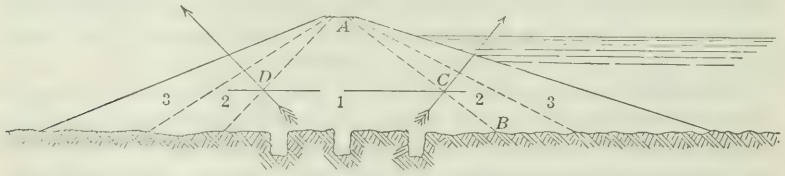


FIG. 21.

At this time, experiments had been made to determine the physical state of the core material, and it had been found that it was possible for two men, working from a boat in the pond, to force a 1½-in. pipe down into the core for a distance of more than 60 ft., demonstrating a condition of semi-fluidity or at least plasticity. This condition suggested to the speaker for the first time a danger that seemed not to have been recognized before, and he immediately called Mr. Schuyler's attention to it. Briefly, it would appear that where the mass of core material (placed in accordance with the designs as practically conventionalized by Mr. Schuyler) is in a semi-fluid or plastic state, it must be regarded as a heavy fluid filling the interior of the dam, and exerting a pressure against the interior of the slopes, not parallel with the base, but upward and normal to a plane corresponding to that zone within the mass at which the character of the material changed from plastic to non-plastic. Fig. 21 will make this clear. Zone 1 is assumed to be semi-fluid or plastic, and Zones 2 and 3 to be a non-plastic material.

It is plain from a consideration of this diagram that as long as the zone of plastic or core material is any wider at the bottom than

Mr. Binckley.

at the top, this condition as regards direction of pressure will exist in corresponding degree. It is also quite plain that the static pressure of the core material considered as a fluid, at Point *D* is dangerously near being in excess of the corresponding weight opposed by the overlying mass of slope material above Point *D*, even if this material is of a much higher specific gravity. Such a condition would threaten the disruption of the dam from internal pressure alone, quite aside from any question of sliding due to horizontal pressure. In the case of the Necaxa Dam, although the speaker believes that Mr. Schuyler was impressed with his suggestion, it was too late to make any change in the design of the dam, as the work was well advanced, and, as a matter of fact, there had been no alarming symptoms. Unfortunately, on account of the scarcity of suitable material, the placing of some very light rock in the up-stream slopes was subsequently permitted. This rock, as the speaker understands it, was practically the same as the well-known "caliche" of Mexico, and was very much lighter than the material previously used. This fact, coupled with the rapid draft of water for power purposes, which lowered the level of the reservoir and removed the support of the water from the upper slope, resulted in what may properly be described as the explosion of the dam. The internal pressure, in amount and direction, was too much for the resistance of the up-stream slope and it went out into the reservoir as did the up-stream slope of the Calaveras Dam.

Shortly after this accident the speaker met Mr. Schuyler in British Columbia, and the former conversation on the subject was recalled. He stated at that time that thereafter he would always limit the impervious core material to a relatively small cross-section, and if he had lived to construct other hydraulic-fill dams, there is not the slightest doubt that he would have corrected the error that he had come to recognize as existing in previous designs; and certainly it is hard to understand how, with the very plain lesson taught by the Necaxa case, the same error could have been committed in the Calaveras Dam, with the aggravating circumstances of slopes of dry fill and of tongues or strata of clay lubricant running out nearly to the slopes, as described by the author.

As regards the Coquitlam Dam, referred to by Mr. Hazen, the material in the core was of a very different character from that commonly found in hydraulic-fill construction. This material was a blue, glacial, so-called clay—actually a very fine silt, having rather remarkable qualities. It is a material with which the speaker became familiar a great many years ago while constructing a dam on the Puyallup River, and is the same as the bulk of material handled in the Seattle re-grade operations. Fine as this material is, and much water as it holds, it has the quality of setting very firmly in a short time. Indeed, its qualities are exactly opposite to those of quick-

Mr. Binckley.

sand, hence neither the speaker nor Mr. Schuyler had any hesitation in using it as a core material for the Coquitlam Dam. This work in collaboration with Mr. Schuyler, was designed and located by the speaker who conducted all the test-pit and tunnel explorations and the stripping of the site, etc. This structure was about 700 ft. long on the crest, was provided with a surface of heavy rip-rap laid in cement mortar both up and down stream, and was protected against overtopping by a spillway 15 ft. deep and, as the speaker remembers it, 250 ft. long through solid rock. For the greater part of its length, the dam rested on the same hard, tight, "glacial clay" already described, abutting on the east end against the diorite ridge in which the spillway was cut. As originally designed, this dam was perfectly safe, and any additions to its section were not improbably made in deference to the same element of the population of that district who opposed the construction of the dam under any conditions. It is possible, however, that a scarcity of suitable material for the heavy slopes developed during construction, in which case, of course, such increase of section as that actually given would have been wise.

In view of the costly lessons of failure and accessible experience in the accepted technique of construction, the desiderata for good design and safe construction of an hydraulic-fill dam would seem to be:

1. Plenty of coarse and heavy material in the slopes, deposited by flowing water, and segregated in such a manner as to form a progressively dense mass from the slopes inward toward the center.

2. The provision of adequate muck trenches or cut-off walls along the zone of the impervious core.

3. The careful avoidance of the introduction of any lubricant in the mass of the slopes.

4. The construction of an impervious core containing enough granular material to destroy or greatly reduce its plasticity, while, at the same time, containing enough clay to secure an impervious or substantially impervious core.

5. The provision of ample spillway capacity to guard against the possibility of the dam being overtopped, and in case of the slightest doubt, to provide a down-stream slope surface capable of carrying, without danger of erosion, a considerable overpour for a few hours.

Mr. Paul.

CHARLES H. PAUL,* M. AM. SOC. C. E. (by letter).—The writer is strongly in favor of comparatively narrow cores for hydraulic-fill dams. In several cases which have come to his notice, an unfavorable condition has been introduced by the fact that, in beginning construction, the core width at the bottom was allowed to be so great that the stable material in the outside slopes necessarily overhung portions of the core which had not yet consolidated. This is liable to result in sloughing of

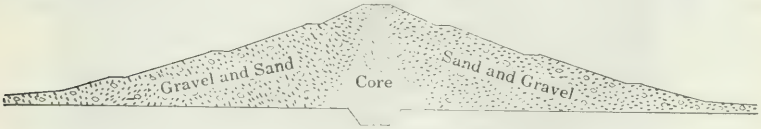
* Dayton, Ohio.

the coarser material into the core, thereby weakening the structure by permitting the core material to encroach still further into the region of the outside slopes. Mr. Paul.

In designing the dams for the Miami Conservancy District, the width of core at any point was limited to the height of the structure above that point. This gave a maximum width of core at the bottom of not more than one-fifth the bottom width of the dam, whereas, in many cases, hydraulic-fill dams have been built with core widths as great as one-third or one-half the bottom width of the dam. Comparison of the wide and narrow core is illustrated in Fig. 22.

One of these sketches, Fig. 22 (*a*), shows a core about one-third the width of the dam at the base, and it may be seen that before construction has progressed very far, the material in the outside slopes will actually overhang the core material. This is an undesirable condition. In the other sketch, Fig. 22 (*b*), the core width at any point is about

Type with Wide Core, Sides Subject to Falling into Core.
(*a*)



Type with Narrow Core, Sides of Superior Stability.
(*b*)

FIG. 22.

equal to the height of the dam above that point. There is very little overhanging of the coarse material, and under such conditions sloughing of the coarser material into the core is practically eliminated.

In a design with the core width at any point equal to the height of the dam above that point, there is no difficulty whatever in obtaining a cross-section stable against sliding and with side slopes as steep as will be permitted by considerations other than the lateral pressure of the core. This point also was carefully worked out in the designs of the Conservancy dams; as a matter of fact, the side slopes of those dams were determined by other considerations.

Fig. 23 is a view of Huffman Dam under construction, showing the comparative width of the core and the volume of stable material in the outside slopes. Dredge pumps located at the toe of the dam (at the stiff-leg derrick) handle the sand and gravel obtained from the

Mr. Paul. river bottom, pumping it up into the embankment. The hydraulic giant in the foreground is knocking down the clay overburden on the hillside to supply the deficiency of core material. The concrete structure in the distance is a retaining wall forming one side of the outlet structure. The shape of this wall fits the final cross-section of the dam.

There is danger, however, of making the core too narrow, and the writer is inclined to believe that to follow Mr. Hazen's recommendation and make the core "as narrow as possible," or as illustrated in Fig. 3, would not be wise. It is not possible, in construction, to control the width of the core absolutely. Some leeway should be allowed, so that when the coarser material encroaches on the core for reasons beyond the control of the men on the job, there will be no danger of its shooting entirely across the core and resulting in a tongue of coarse material extending all the way through the dam. Also, there is a transition zone between the coarse material and the true core material, which must not be overlooked. The writer is of the opinion that a happy mean, between extremely wide and extremely narrow cores, is the proper thing, and he believes that the design adopted by the Miami Conservancy District is a conservative one from both viewpoints.

As to the gradation of particles in the core itself, several analyses have been made, from time to time, by the U. S. Bureau of Soils, at Washington, D. C., of samples taken from the cores of the five dams now under construction in the Miami Valley. Table 5 gives a typical analysis of the core material from each of these dams.

TABLE 5.—Typical Analyses of Core Samples.

	Bureau of Soils, standard sizing, in millimeters.	Germantown Dam. Percentage.	Englewood Dam. Percentage.	Lockington Dam. Percentage.	Taylorsville Dam. Percentage.	Huffman Dam. Percentage.
Fine gravel	2 to 1	0	0	0	0	0
Coarse sand	1 " 0.5	0	0.4	0	0	0
Medium sand	0.5 " 0.25	0	0.3	0	0.1	0
Fine sand	0.25 " 0.10	4.2	4.6	1.0	1.9	2.8
Very fine sand	0.10 " 0.05	17.3	19.6	10.1	18.1	15.0
Silt	0.05 " 0.005	56.0	52.7	59.6	59.7	52.5
Clay	0.005 " 0	22.5	22.6	29.3	20.2	29.8

It will be noted that although there is some difference in the gradation, yet, taken as a whole, the samples are after all remarkably similar. This in spite of the fact that each of the dams is in a different river valley, two of them (the Germantown Dam on Twin Creek, and the Lockington Dam on Loramie Creek) being 50 miles apart. A sample of Calaveras core, analyzed by the same methods gave very different results, as it was composed entirely of material finer than 0.05 mm., and 60.1% of it was finer than 0.005 mm. Undoubtedly, a

Fig. 23.—View of Huffman Dam Under Construction.

core of such material would behave quite differently from those represented by the analyses given in Table 5. Mr. Paul.

At the Taylorsville Dam there is an excess of fines in the material available for hydraulic fill, and a considerable percentage of the fines in the core are being wasted. On the other hand, at the Germantown and Huffman Dams, the borrow-pit material is deficient in fines, and additional fines are being sluiced in from the glacial till on the hillsides, to supplement the gravel and sand from the river bottoms. It is a source of satisfaction to the engineers concerned that, in spite of these necessary manipulations of the raw material, it is possible to secure the comparatively uniform results shown in Table 5.

Serious construction difficulties would be encountered at all the Conservancy dams if the limit of fineness of core material was to be held to 0.01 mm., as suggested by Mr. Hazen. Moreover, the cost of construction would be greatly increased, and presumably the imperviousness of the core would be considerably lessened. It should be said, however, that care is being taken not to have a large percentage of the very finest grade, namely, material finer than 0.005 mm. Some of this, it is believed, is quite desirable.

Satisfactory consolidation in the cores of all these dams is taking place, regardless of the fact that about 40 to 50% of the core material is finer than 0.01 mm. This is evidenced by rod tests, ball tests, and pressure-cell readings, as will be shown later.

The writer sees no great advantage in immediate consolidation of the core. Considerable time may be allowed for this process, because of the fact that during construction the height of the dam, and, therefore, the pressure from the partly consolidated core, does not increase greatly from month to month. Provided a reasonable rate of consolidation is assured, the writer feels that there is no need of going to extremes, or sacrificing water-tightness, to secure a more rapid consolidation.

The procedure being followed to secure satisfactory cores in the Conservancy dams and the methods of studying action of the material in the cores have been described in an article entitled, "Study of Pressures in Hydraulic Dam Cores."* This article describes in detail the Goldbeck pressure cells, several of which are in use in the dams of the Miami Conservancy District, for determining pressures in the cores as construction progresses, and particularly for determining the relation between vertical and lateral pressures at various elevations. About three years ago, the writer had occasion to make an investigation of the operation of the Goldbeck pressure cells under various conditions, and outlined with Mr. Goldbeck a series of tests which were conducted by the U. S. Office of Public Roads for the purpose of showing whether or not these pressure cells would give reasonably correct results if

* *Engineering News-Record*, December 25th, 1919.

Mr. Paul. installed in hydraulic-fill dam cores. Rather elaborate experiments were made, which have been referred to elsewhere.*

The writer is satisfied that this apparatus is the best instrument that has yet been devised for determining the rate of consolidation and for making pressure studies under such conditions. Inasmuch as a full description of this pressure cell has been given by Mr. Goldbeck (page 1764), and also in the *Engineering News-Record* articles previously mentioned, the writer will not take the space here to describe the construction of this cell in detail. Very briefly, it is a closed flat circular box, 5½ in. in diameter (similar to a large shoe-polish box, in shape), the top or bottom of which acts as a movable diaphragm, operated by admitting compressed air, through a small pipe, to the inside of the box. Air is supplied from a small tank which may be charged with an ordinary tire pump. The box being buried in the material to be tested, the movable diaphragm is held down (or in) by the pressure of the material against it. When the pressure of the air admitted to the inside equals that of the material outside, the diaphragm is lifted slightly, an electric contact is broken, and an indicator light in the registering apparatus goes out. The pressure gauge on the air line, at that instant, indicates the pressure of the material against the movable diaphragm. The operation of the cell in taking such readings requires a movement of the diaphragm of less than $\frac{1}{10}\frac{1}{000}$ in. It works so positively that, by proper manipulation of the air feed, the light may be turned on and off any number of times without moving the pointer on the pressure gauge. The cell may be set horizontally, vertically, or at any angle, and thus may record the pressure from any direction. In this manner, valuable comparisons between vertical and lateral pressures may be secured.

There is no comparison between the operation of this pressure cell and the crude foundation tests referred to by the author, and even if they were comparable, the recorded pressures of the Goldbeck cells would be higher than the actual pressures, rather than lower, because of the principles of operation just described. Whether the actual readings are absolutely correct is not vital; the comparative readings are of great interest and value, and, to those who have used the apparatus, give promise of yielding information of considerable importance.

It would be a mistake to attempt to anticipate the conclusions which will finally be drawn from the complete record of pressure-cell tests and other core studies in the Conservancy dams. This subject was investigated very thoroughly before the construction of hydraulic-fill dams in the Miami Valley was undertaken, and extensive studies of the behavior of the core material in these dams have been carried on during the two years or more since construction began. Pressure cells will be installed at all the dams in their maximum sections. This in-

* *Engineering News-Record*, April 18th, 1918, and December 25th, 1919.

stallation at one of the dams has been carried to the extent where interesting results are already available. However, the data will be so much more complete after another year or two that it seems premature to attempt to draw final conclusions from those now at hand. With that limitation, however, the writer feels that the results obtained up to this time are worth presenting. Mr. Paul.

At the Germantown Dam, construction has been carried to a point 60 ft. above river level. This dam will have a maximum height of 110 ft. Pressure cells have been installed in the core at 10-ft. vertical intervals for determining lateral pressures, and at 20-ft. intervals for recording vertical pressures. The results obtained to date are shown in Fig. 24.

There is every indication from these results that the core at the lower elevations is consolidating in a satisfactory manner. The lateral pressures at the lower elevations are considerably less than the vertical pressures, and the diagrams indicate that progressive consolidation is taking place from month to month. At the present time, in the lower portion of the core, the lateral pressures are only about two-thirds of the vertical pressures. At the upper elevations, vertical and lateral pressures are about equal, confirming the generally accepted belief that, at these upper elevations, the core is still a perfect fluid. Viewed as a whole, the readings obtained so far at the Germantown Dam appear to be reasonably correct and suggest no reason for doubting that the pressure cells are giving approximately reliable data. There is also every indication that the core is consolidating rapidly enough for ample safety in construction. Similar data in complete form from all the dams will be available at a later date.

At the Taylorsville Dam, it has been possible to secure actual samples of the core at considerable depth from the surface, after pumping on that section of the dam had been discontinued for the winter. These samples, although they contain perhaps 50% of material finer than 0.01 mm., show without possibility of doubt that the core is solidifying at a satisfactory rate. In fact, at the present time at Taylorsville, it is possible to walk across the core without difficulty, and it has been decided before resuming work on this section of the dam to plow up the core, in order to be sure of obtaining a good bond for the new work.

Penetration tests with the 6-in. cast-iron ball are being made regularly at all the Conservancy dams. Fig. 25 shows the record of ball tests at the Germantown Dam.

The ball sinks to a maximum depth of only about 16 ft., while at the Calaveras Dam the writer is informed that in many cases penetration of the 6-in. cast-iron ball was as great as 50 to 60 ft. Several tests have been made by leaving the ball in place for a period of several days in order to see whether it would gradually continue to settle,

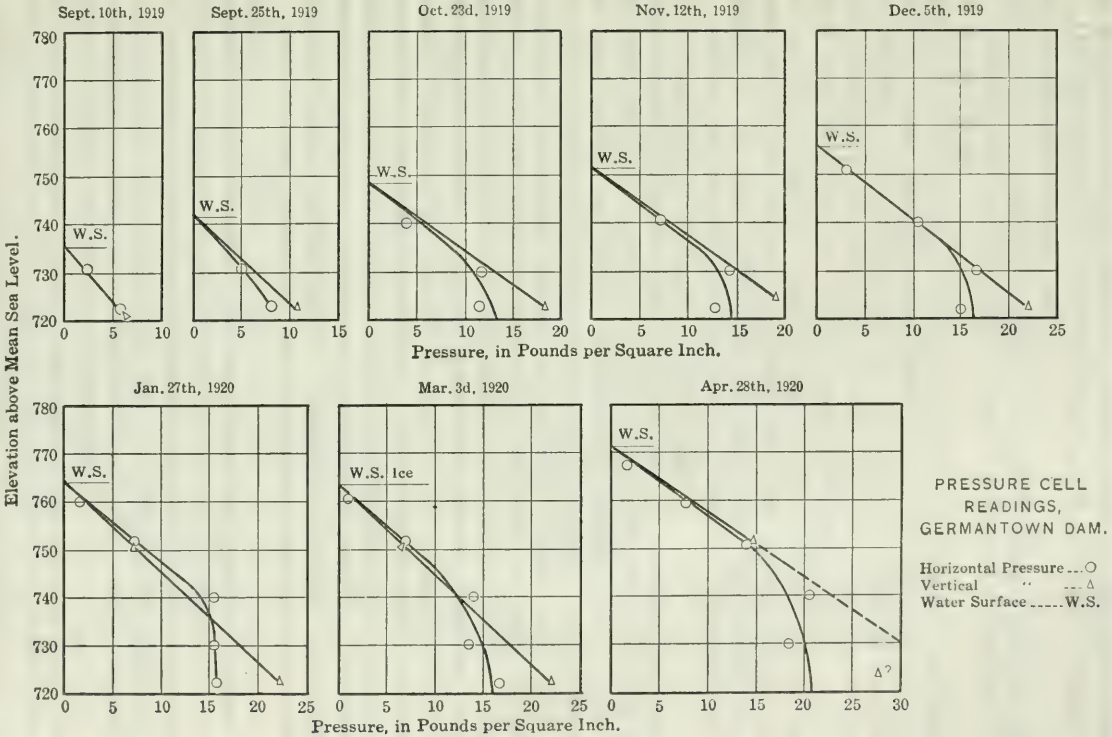
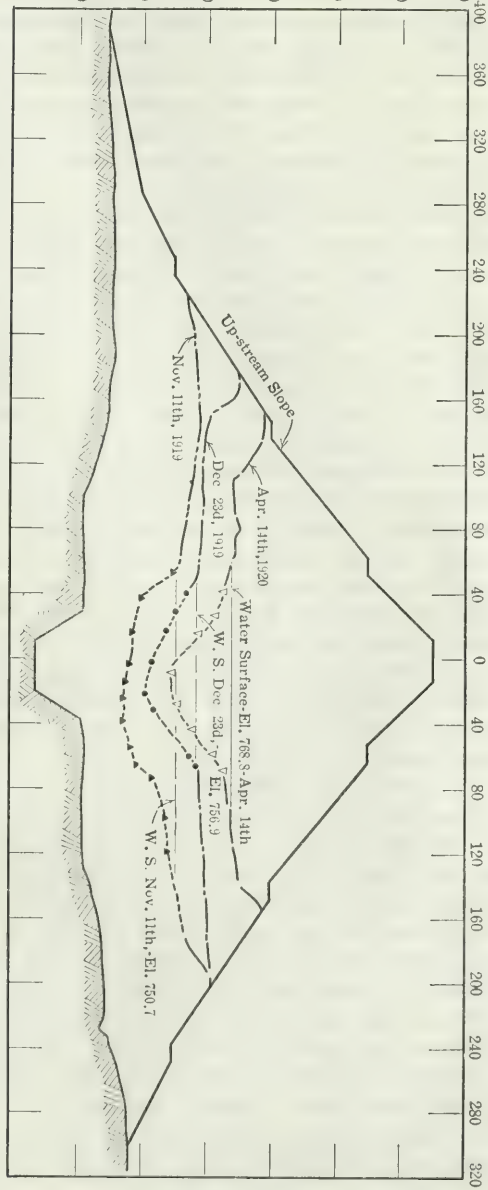


FIG. 24.

Mr. Paul.

Mr. Paul.



PENETRATION TESTS—6 INCH CAST IRON BALL

GERMANTOWN DAM.

Fig. 25.

Mr. Paul. but it has been found that after the first minute the amount of settlement is generally negligible. It is true that material which will support a 6-in. cast-iron ball is not necessarily very stable, but, on the other hand, it is definitely along the road toward consolidation. It can hardly be called a fluid. Moreover, these ball tests are valuable, owing to the fact that they are being used in other hydraulic-fill dams, and therefore, give means of comparison at dams in various parts of the country. The great difference in ball penetration at Calaveras and at the Conservancy dams, is significant.

Rod tests have also been made at the dams in the Miami Valley, but little value is attached to the results obtained by these methods, as the personal element has such a marked effect on the results. There is nothing very definite about a penetration obtained by two men, working from a boat, with an iron pipe or rod 30 or 40 ft. long. These rod tests, however, do confirm the indications shown by the pressure-cell tests that a satisfactory consolidation of the cores at the lower elevations is taking place. The writer's experience does not confirm the author's statement that material too stiff to be penetrated by the rod will still exert full lateral pressure. This may be disproved by working the rod test in a natural clay bank.

It is interesting to note in this connection, also, that maximum rod penetrations in the Conservancy cores do not exceed about 30 ft., while, at Calaveras, the rod penetrations are reported to have reached depths of from 80 or 90 ft.

The author states that material with 50% voids "is capable of standing up in a boring driven beyond the casing for several feet" and, in another paragraph, that this same material "exerts full lateral pressure, corresponding to its height and weight per cubic foot." The two statements appear to be inconsistent. There is a question in the writer's mind whether the percentage of voids in such material is necessarily a measure of stability. He doubts whether void determinations from boring or other samples are reliable, even as representing the true condition of the core itself, because it is so extremely difficult to bring up a sample in which the moisture content or conditions of compactness has not been disturbed. This is a point which is often disregarded, but which, nevertheless, is very important. The very act of taking a boring sample, especially in such material, tends to compact the sample and change its moisture content.

To one who has not made a study of the history of hydraulic-fill dams, the impression might be gained from Mr. Hazen's paper that many such dams have failed. Such is not the case. Aside from the two failures which have been mentioned, namely, Necaxa and Calaveras, both of which have been fully accounted for, the writer has been unable to learn of any other instances of serious difficulty with

dams built by approved hydraulic methods. On the other hand, there are many such dams in service, throughout the country, at which there has been no suggestion of trouble. In fact, it is apparent that, in a hydraulic-fill dam, construction conditions are generally more severe than those which will be encountered in service; a decided advantage for that type of dam. Mr. Paul.

JOHN E. FIELD,* M. AM. SOC. C. E. (by letter).—The writer was glad to see Mr. Hazen's paper, because heretofore the hydraulic method of construction for earth dams has seemed to be in disrepute, not only with the general public, but among engineers. This disrepute is the result of the failures of the Necaxa and the Calaveras Dams, in particular, and to some other failures not so well known. The failures have naturally attracted more attention than the successes, of which latter there are many. There are in Colorado alone three successes, namely, the Terrace, Santa Maria and Onion Valley Dams, and one failure, the Schaefer Dam, near Florence, Colo. In the case of the Schaefer Dam, the material was entirely unsuitable, being either of yellow clay, or rough angular sand rock, with very little sand or loam. The clay puddle remained the consistency of cream, and when the dam was finally finished by steam shovel and train, the central puddle material was allowed to rise as the dry fill progressed, and finally was drawn off. Mr. Field.

In considering the phenomena of engineering failures, there is generally a difference of opinion, and some mystery; especially is this true of the failure of earth dams. Doubtless, much valuable information and data are suppressed in cases where litigation over damages is in progress. The absence of trained observers before and during the time of failure, and the unreliability of testimony by the casual, but honest, observer, make these differences of opinion possible. In the case of the failure of the Calaveras Dam, the observations are unusually complete and reliable, and its discussion by the Profession should be thorough. This is the excuse for the random comments here presented.

In discussing hydraulic-fill dams, the very different conditions and objects to be attained must be kept in mind. Recently published articles on hydraulic-fill dams have considered particularly the construction period. The troubles at Necaxa and Calaveras were not properly dam failures, and to the general public were of little interest, as they involved neither its life nor property; the owner and the engineer were intensely interested and the latter has given considerable prominence to the phenomena presented. It is better to have one dam in ten fail during construction, than to incur the enormous expense necessary for all in order to insure against the one failure. On the other hand, the failure of one dam when in use involves such great and

* Denver, Colo.

Mr. Field. widespread consequences that any extra precaution and expense is justified to insure the safety of all.

In the failure of earth dams by slipping that have come to the writer's attention, one cause has seemed to predominate, namely, excessive width of core at the base in the case of hydraulic fills, and poor foundations or permeable base in the case of other dams. The slips at the Wachusetts Dam and at Standley Lake near Denver are examples of the latter.

The writer has read the published accounts of the failure at the Calaveras Dam, and has talked with some engineers who visited the dam before and after such failure. The cause seems to the writer to be quite certain, that is, a layer of fluid material existed under a portion of the upper toe. Whether this layer extended to, and connected with, the puddle core proper is not important. The weight of the dry fill or of the core, or both, transmitted pressure through the layer of fluid material to the lower portion of the upper slope, probably finding the weakest point at or near the upper edge of the concrete paving, for Mr. Hazen has made the following statements:*

"The lower part of the toe had been paved with concrete and this extended 13 ft. above the water level. The top of the concrete formed a conspicuous line and observers say this seemed to rise several feet. * * * * saw the tower 230 ft. high swaying back and forth for a moment, after which it fell in a direction straight away from the dam. * * * * The concrete slope (June 18th, 1917) lifted and cracked parallel to the upper edge and 6 ft. below it for a distance of 700 ft., more or less across the dam."

It appears that a lift of the face near the upper edge of the concrete preceded or was simultaneous with the horizontal movement; whether there was an upward movement of the surface of the fill below the water line is not known, and it may be that the water pressure threw the minimum effective weight of the toe just above the water line, and perhaps also the water caused the puddle on the upper toe in the base of the dam to remain more fluid and to drain less readily than the lower toe which, although lighter in section, did not fail, and the water also saturated and rendered less stable the supporting upper toe. The fact that so many slips in dam enbankments have been on the upper slope, and at a time when water was in the reservoir, leads the writer to suspect that that condition is a contributing cause to failures. The objects of letting water rise on the fill are two: first, to help support the upper toe, and, second, to make use of the water so stored. If the water is drawn off for use, then the first object is lost. In fact, the drawing down of the water may be a menace, as it would leave a more or less saturated upper slope of less stability to support the core pressures.

* *Engineering News-Record,* April 4th, 1919.

This phenomenon is most marked in caving river banks when floods begin to recede. Mr. Field.

It is to be remarked that the failures both at Calaveras and Necaxa were toward the flatter slope, up stream. In the first, there was water in the reservoir, while in the latter there was none at the time of failure, but there had previously been water against the upper face.

It is only natural that in the process of construction the puddle material should extend farther up stream from the center line under the flatter slope than under the steeper lower slope, unless such a result is guarded against. The engineer building any dam, naturally and properly has in mind the safety of the work when in use, rather than its safety during construction. For the completed hydraulic fill, he strives for a well drained lower toe of as great cross-section as is practicable. He also strives for an impervious core of ample thickness, but for the upper toe he desires only a protective coating and support for the puddle core. In ordinary fills the farther up stream the impervious core is, the better. All these considerations tend to place the puddle up stream and under the flatter upper face, and this, if carried too far, would result in failure during construction.

In construction, it seems wise to predetermine the position and minimum thickness of the puddle core and to give the greater stability to the upper toe, especially where water is permitted to rise in the reservoir. The presence of excess width of puddle material in the lower portions of the Calaveras Dam would account not only for the failure, but for the discrepancy in the coefficients of friction as determined by experiment at San Pablo, and as indicated by the failure at Calaveras. The planes of rupture formed by settlements prior to the strengthening of the upper toe were not eliminated by such strengthening, and remained a source of weakness not mentioned by the author.

Mr. Hazen states, referring to hydraulic fills, "experience does not show that the material is graded according to the theoretical section." The coarse materials are deposited near the toes and then filled with finer materials, making a very heavy compact mass with a low percentage of voids. In the experience of the writer, toes so placed were sufficiently impervious during construction to prevent appreciable seepage through them. In practice, also, the sand settles immediately on reaching the water of the pool, and rapidly pushes toward the center, and with each successive raising, and the occasional lowering of the water in the pool, the effect is somewhat as shown in Fig. 26. The points of sand extending into the puddle aid in the drainage.

The minimum thickness of the puddle is the minimum distance between points of sand touching the area, *A*, of the diagram, and the maximum thickness of the puddle is the maximum width of puddle within areas, *A*, *B*, and *C*. In calculating the weights of toes, the

Mr. Field.

minimum thickness of core should be taken, while if the sand and puddle are incorporated by artificial mixing, the thickness of the impervious diaphragm may be taken as the maximum width of the puddle.

In connection with the drainage and saturation in hydraulic and ordinary dams, it may be of interest for the writer to state that at the Terrace Reservoir he had occasion to excavate into the puddle core and also into the portion of the dam placed by teams. The lower 170 ft. of the dam was hydraulicked, while the upper 30 ft. was placed by teams and wagons and thoroughly rolled. In the excavation mentioned, the puddle core was nearly as stiff as material used for making stiff mud brick, and the consistency of the team fill was the same, as far as could be determined by handling and appearance, showing that the team fill had taken on moisture, and the hydraulic fill had lost moisture, until they had each apparently reached a common degree of saturation. Within the line of saturation, one should expect to find similar degrees of saturation in either class of dam.

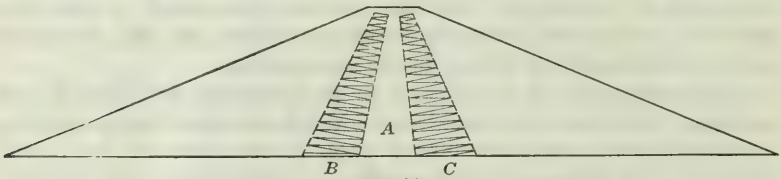


FIG. 26.

At the Terrace Reservoir, the water used in sluicing, particularly during low water in the river, contained a very appreciable quantity of alum in solution, so that it was perceptible to the taste, and its use for domestic purposes was avoided. This fact may have had something to do with the very satisfactory settling of the core material. While fluidity in the puddle increases with depth, increased depth also aids drainage, for under the high pressure present in high dams the water is forced out laterally and the material is consolidated. In the hydraulic fill, the pressure is gradually increased, and the drainage and consolidation are completed in the lowest layers first, the upper layers adjusting themselves as the shrinkage, due to loss of water, progresses. In this matter, the experiments and tests being made by the engineers of the Miami Conservancy District are enlightening.

The author says:

"If drainage and consolidation of the core are not secured, however, safety can only be reached by making each toe large enough to stand the full pressure of the liquid core, and, in that case, in order to be safe, a dam built by the hydraulic method must have flatter slopes and a larger section than is otherwise needed for an earth dam."

This statement is obviously true if one grants the premises for the hydraulic fill, and considers only the dimensions necessary to resist

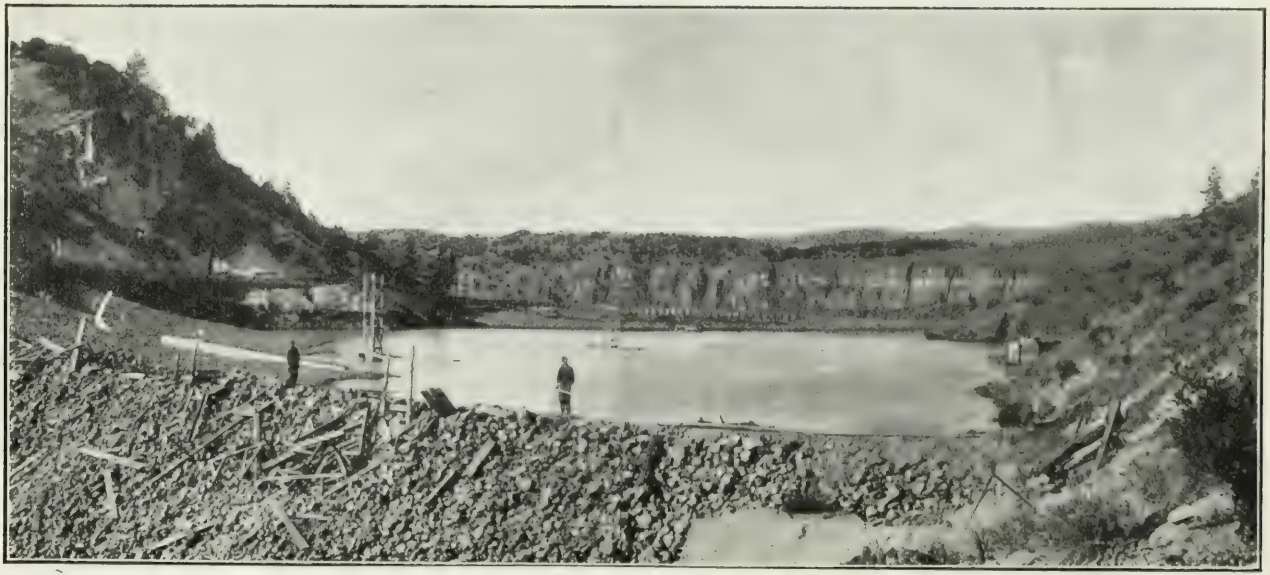


FIG. 27.—TOP OF TERRACE DAM, LOOKING NORTH (UP STREAM), SEPTEMBER, 1910, SHOWING CHARACTER OF TOE MATERIAL, WASTE WELLS AND SETTLING BASIN.

sliding and percolation for the dry fill. In earth-fill dams, however, the slopes, in order to stand, must be at least 1½ to 1, but in order to keep the lower slope outside the line of saturation, it may have to be increased materially, and to have a firm stable bed for the rip-rap facing in the upper slope, it too must be increased. With loose rock rip-rap, a slope of 2½ to 1 is none too flat, and a slope of 2 to 1 for the lower slope is necessary, as 1½ to 1 slopes are not stable under the action of the elements. We may assume, in most cases, that the minimum section of an earth dam is a base five times the height plus the crest width, and it remains only to consider whether such a section would be sufficient for an hydraulic fill. Mr. Field.

The writer wishes to comment on that portion of the paragraph quoted, which reads "If drainage and consolidation of the core are not secured, * * *". It is his opinion, first, that with the material generally found, the core does drain and consolidate, and will be self-supporting, and second, where puddle material is not satisfactory, that drainage and consolidation can be secured. Narrowing the settling basin and wasting the material, as suggested, should be avoided where material is as scarce and difficult to obtain as at the Terrace Dam in Southern Colorado. The scattering of coarse material over the surface of the settling basin would aid in both the drainage and consolidation of the core. The core material should be homogeneous, but need not be uniform in size. The ideal condition would be a mixture giving the greatest weight per unit, the greatest impermeability, and the most stable character. Such a material would be a mixture of coarse and fines, as in concrete aggregate. The writer believes that the presence of large boulders in the core is no detriment; that in time such boulders placed in the core would settle to a point where the core would be quite stable. Mr. Hazen states that "material with 50% of voids in a considerable depth, acts essentially as a liquid." The settling of boulders through and into such material would probably entirely overcome the increasing instability due to depth.

However, in practice there should be added to unstable core material, and thoroughly incorporated therein, boulders, gravel, and sand. The method of adding these coarser materials need not be expensive, and will vary according to the plant, equipment, and local conditions. If the minimum thickness of core is first determined, and then measures are taken to secure stability, there need be no material widening of the base beyond that required for other methods.

The author states that "If a factor of safety of 2 is required, the outside slope must be 4 to 1" [on the basis of a coefficient of friction of 0.5 and equal weight of core and toe material] "but in view of the records of slips in actual dams, it may be well to take a higher factor of safety." With this conclusion the writer cannot agree, for the following reasons:

Mr. Field.

First.—As the coefficient of safety is to apply only to the period of construction, a factor of safety of 2, or even less, may be assumed, as the consequences of failure do not involve the public safety, but a factor of 3 would involve very greatly increased expense. At Necaxa, with slopes of 4 to 1, the volume in the fill would have been doubled for the lower toe, and 33% greater for the upper toe, or the 2 000 000 cu. yd. would have been increased far more than was lost by the slip. The same is true of the Calaveras failure, but even on slopes of 4 to 1 and assuming too wide a puddle at the lower levels, there is no assurance that the failures would not have occurred.

Second.—The lower and lighter toes at both Necaxa and Calaveras did not slip.

Third.—The weight of material in toe and core is not equal, but the two materials as suggested by Mr. Hazen are generally as 5 to 4. Assuming that the toe material has a weight of 120 lb. per cu. ft., and the puddle material 100 lb. per cu. ft., with slopes of $2\frac{1}{2}$ to 1, and a coefficient of friction of 0.5, the factor of safety would be 1.5, which seems to be sufficient for the construction period.

Fourth.—Because defects in execution are revealed in the records of the failures, and may now be avoided.

Fifth.—In the increasing of the factor of safety, Mr. Hazen increases the slopes, but increasing the slopes has a tendency to widen the puddle core and extend it under the slopes, which seems to have been the cause of failure at Calaveras.

The writer accepts generally the principles enunciated by Mr. Hazen, but arrives at somewhat different conclusions, as follows:

1.—To limit the size of material to 0.01 mm., about $\frac{1}{2\,500}$ in., would not necessitate wasting more material than would naturally be wasted in the process of construction, under ordinary conditions.

2.—The puddle should be at a minimum, especially in the lower portion, and care should be taken to have the toes of full width, particularly at the base of the dam.

3.—The study of borings and the measurement of vertical and horizontal pressures, as on the dams being built by the Miami Conservancy District,* are essential.

4.—An ample factor of safety during construction seems to be 2, as the added volume due to a greater factor of safety would generally be greater than the probable loss should failure occur.

5.—Insure the solidity of toes by the use of rock fill placed hydraulically or otherwise. If placed hydraulically, greater weight will generally result.

6.—It is, in the writer's opinion, unnecessary to provide against quicksand conditions.

* *Engineering-Contracting*, June 16th, 1920.

Mr. Field.

A strict application of the principles laid down would probably not reduce the number of hydraulic dams, since their observance would increase the confidence of the public and the Profession in them, although a few, where clay was practically the only available core material, would not be undertaken. Observance of these principles would not materially increase either the volume or cost, but with proper testing would eliminate present uncertainties, and result in the more extensive use of this most useful and economical method of dam construction.

Mr. Hazen states that a 6% slope was necessary at the San Pablo Dam for sluice flumes with wooden bottoms. At the Terrace Reservoir the flume had a grade of 2.9 ft. per 100, and was lined with 6-in. wood blocks, much rock weighing from 50 to 100 lb. was carried, and some was as heavy as 200 lb. The rock was in part rounded, and in part angular; these flumes gave as good carrying efficiency as those lined with steel on the same grade, and were much more economical.

It is to be hoped that further investigations and experiments will be made to determine whether the suggested minimum size of 0.01 mm. for puddle material cannot be very materially increased or decreased with safety; whether the core material, even though of clay, does not drain laterally and consolidate so as to become self-supporting; and whether it is wise to allow water to rise in the reservoir as the fill progresses.

Mr. Holmes.

J. Albert Holmes,* M. Am. Soc. C. E. (by letter).—In this exceedingly interesting and valuable paper, Mr. Hazen has demonstrated the importance of a narrow central pool in hydraulic-fill dam construction and the influence a narrow pool is likely to have on the grain size of the core material.

As a result of his study, he proposes as a minimum for core material an effective size of 0.01 mm., and to waste from the pool any particles finer than this size. The writer is greatly interested in both these features and believes in their effectiveness in producing a stable and impervious structure. He has, however, brought about these conditions in a somewhat different manner.

In Table 2, Mr. Hazen has listed two dams, with one of which the writer was connected as Resident Engineer, and with the other as Resident Engineer for the Consulting Engineers.

The Somerset Dam, at Somerset, Vt., is in the region of the northern ice fields, and the material of construction is a boulder clay. The Paddy Creek Dam is one of a group of four dams at Bridgewater, N. C., and is built of the decayed and disintegrated rock of a non-glacial country.

* Decatur, Ill.

Mr. Holmes.

The method of construction was semi-hydraulic, that is, the material was excavated by steam shovel, loaded into dump cars, hauled to the dam site by steam locomotives, dumped from trestles and dikes, and sluiced into a central pool where segregation and sedimentation took place.

At Somerset, the sluicing water was pumped from the reservoir and delivered through pipe lines laid along both dikes, with hose connections at frequent intervals. The previously loosened material in the dikes did not, of course, require the same quantity or velocity of water necessary to excavate hydraulically and transport material to a straight hydraulic-fill dam. The quantity of water used at Somerset was about 1 sec-ft. as against 15 to 20 sec-ft. at San Pablo.

To regulate the height of pool at Somerset, an overflow was provided. During the first stages of the work there was a discharge from the pool, the flow carrying with it an estimated quantity of 100 cu. yd. of fine material per 24 hours. It was sought to retain this material, rather than to get rid of it, and during much of the time there was no overflow from the pool and the fine particles were retained. That this could be safely done was due to the character of the borrow-pit material, the narrow pool and core, and the wide supporting dikes. Before the pool was formed, the outer dikes were built up by dumping, first from the trestles and then over the edge of the dikes.

At Somerset, it was desired to give the core a broad base, covering the middle-third of the dam and making seepage travel long at the junction of the core with the stripped surface. There was no cut-off trench. After forming the broad base the core was narrowed quickly so that at completion it occupied one-fifth the cross-section.

The pool width varied from about 60% of the width at the base of the dam, to less than 30% at the top. Sluicing from the dikes into the pool left all gravel and boulders in the dikes. The size of the rock going into the dam was limited only by the capacity of the shovels to load it.

The dikes were watched carefully for movement during construction, but no movement was detected. They were broad and stable at all stages of the work. A complete mechanical analysis was not made of the Somerset core, but individual samples showed 94% passing a 200-mesh sieve, the maximum size being about 0.10 mm.

The core of this dam would probably be placed by Mr. Hazen in the 0.001 to 0.002-mm. effective size class. The dam has been in use nearly seven years and is perhaps entitled to be listed with "A Number of Earth Dams That Have Stood Successfully."

The material of which the Bridgewater Dams were built contained very little that could be classified as clay, and the deeper the excavation was carried in the borrow-pits, the coarser the material became.

Mr. Holmes.

The problem here was to get enough fine material to form the cores. On the other hand, the disintegrated rock did not furnish sound and durable gravel and boulders to give weight and permeability to the outer dikes, although the resulting structures were in every way satisfactory.

The Somerset method of dumping from trestles and dikes was followed at Bridgewater. The sluicing was done from scows floated in the pools on which were mounted monitors and pumps, taking the water from the pools and using it over and over. No overflows were provided; the seepage and evaporation losses amounted to about 0.4 sec-ft. and were made up by pumping from the reservoir to the pool. By this method of sluicing, that is, with water being taken from and returned to the pools and without overflows, all the fine material was retained.

Seepage experiments were made with two combined samples, made up of a large number of smaller samples taken from all parts of the core of two of the dams. These experiments gave a seepage rate for one dam of 30 000 gal. per 24 hours and about 14 000 gal. per 24 hours for the other. This is considering only the core of the dam, from which the samples experimented on were taken.

Mechanical analysis was made of the cores of the three larger dams at Bridgewater. The analysis shows very similar material in the cores of all three dams and that the effective size is nearly three times that suggested by Mr. Hazen as a minimum.

TABLE 6.—Effective Size and Uniformity Coefficient of Core Materials.

	Catawba.	Linville.	Paddy Creek.
Size at 60% by weight, in millimeters	0.146	0.150	0.157
÷ Effective size (at 10% by weight), in millimeters	0.026	0.028	0.030
= Uniformity coefficient	5.6	5.4	5.2

The Bridgewater Dams have rock in the down-stream toe; at Somerset there is some rock in both toes. The Somerset method of dumping from trestles and dikes and sluicing the fine material from the dikes to the core, insures stable dikes and ready control of pool width and character of core material, and in the writer's opinion, permits the use of a fine core material.

Mr. Swanitz.

H. W. Swanitz,* Assoc. M. Am. Soc. C. E. (by letter).—This paper on hydraulic-fill dams sets forth in detail certain principles and theories to be followed in constructing earth-fill dams by the hydraulic process. The author treats the problem as essentially one of filtration and cites

* Vallejo, Cal.

Mr. Swanitz.

an elaborate series of tests and experiments involving microscopic work to determine the size of the grains in the central impervious core, as well as certain tests to arrive at the coefficient of friction of the material.

The paper is based primarily on tests made as a result of the failure of the Calaveras Dam of the Spring Valley Water Company, in California. The assumption is made that practically the same laws of capillarity and drainage apply to the very fine-grained clays as to the fine-grained sands. This may or may not be so, but it has been well known for several years among certain members of the Profession who have had to deal with earth dams, that fine-grained clay drains very slowly and that any embankment (as the core-wall of a dam) deposited under water, will remain in a supersaturated condition for a long time, and, moreover, that this material exerts pressure comparable to pure hydraulic pressure due to a fluid of increased density, the density being that of the core mass.

In the construction of any dam, the type of structure, as well as the method to be used in its construction, is usually indicated quite clearly by the following: (1) the materials at or near the dam site; (2) the importance of the function of the dam (that is, whether it is to form storage for domestic water supply, or power development, or flood protection, etc.); and (3) the amount of money which may justifiably be spent in building the dam. With these facts in mind, if an earth dam is the correct type, then the method of constructing it should receive careful attention. From the list of failures of earth dams, and the well known fact regarding hydraulic pressure just mentioned, it would seem that in the case of any important dam of considerable height, which is to be built in a locality where the available material is mostly pure clay, or rocks largely of a clayey nature, the hydraulic method of construction is definitely indicated.

The whole subject of the design and construction of earth dams reverts to the fundamental fact that the engineers in charge must have a thorough understanding of the nature of the materials which are to be used in the dam, and appreciate the practical limitations of these materials when used in large masses, either alone or mixed with each other in various proportions. This knowledge should be obtained and applied before the dam is built.

A hydraulic-fill dam should be composed of materials which, when finally in place in the structure, will be practically water-tight throughout a portion of the cross-section. If this water-tight portion is to be confined to the central part of the dam, it is believed that it will be a distinct advantage to have the up-stream prism of the dam as compact and nearly water-tight as is reasonably possible. This prism will then be more stable in itself and, due to its added weight and solidity, will

Mr. Swanitz.

offer greater resistance to sliding and settlement both during construction and afterward. The prism of the dam on the down-stream side of the core-wall should be of a comparatively porous nature in order to allow an easy means of escape for the water which drains horizontally from the core during construction, and also for any water which may appear later as a more or less permanent seepage through the dam. This down-stream prism plays an important part in the ultimate resistance of the dam, both against sliding and also in case the structure should be overtopped at any time during construction, or later. For these reasons it will be found advantageous to use a rock-fill toe of more or less generous proportions, as well as a heavy covering of a large rock rip-rap over the entire down-stream face. In important structures an ample freeboard should also be provided.

The dam must have slopes sufficiently flat to insure the stability of the structure under all conditions of saturation from the water in the reservoir. In most cases a slope of 3 to 1 up stream and 2 to 1 down stream will insure this, if the correct materials are used and if they are correctly placed.

The spillway of an earth dam must receive careful consideration as to its capacity, location, and method of construction, as it is the final safeguard against destruction in case of an unprecedented flood occurring with a full reservoir. The dam should be built so that no large cracks will form and no undue settlement will occur after completion. Core material must be of such a nature and placed so that it will drain and solidify while construction is in progress, at least to a point where it will not readily transmit full pressure as a hydraulic medium from the core material at a higher elevation.

These conditions regarding the core clearly indicate that one made of pure clay in which much of the material is very fine-grained should not be used, as it will undoubtedly form a mass of supersaturated liquid mud if placed at the ordinary rate of hydraulic-fill construction. This occurred at the Calaveras Dam. The core should be composed of a mixture of clay, sand, and gravel, the percentage of clay being just sufficient to fill the voids. Such a core will become very stiff and resistant on drying out; moreover, it will drain itself and solidify sufficiently during construction to prevent the transmission of hydraulic pressure or undue settlement in the finished structure.

Mr. Hazen points out that more rapid and thorough drainage and consolidation of the core material will be obtained by wasting the finest particles of clay, and the writer agrees with this, especially where the core material consists of an excess of clayey matter. This in effect amounts to abolishing the method of maintaining a comparatively still, standing, "summit pool", which allows of nearly complete deposition of all hydraulicked material. This fact has been appreciated and used

Mr. Swanitz. by some engineers in hydraulic-fill construction for a number of years, notably in the regrading at Seattle, Wash., and other work.

In cases where considerable doubt exists as to the correct method of building an earth dam (especially one of great height, with an important function to perform) the dry-fill method should be adopted without hesitation. This is certainly the case at Calaveras.

The tests and experiments given by Mr. Hazen would no doubt be of some value in furnishing a rough preliminary idea of the stability that might be expected of a structure built with certain materials in various combinations. However, it is not believed that in the majority of cases it is necessary to go to this trouble.

Mr. Justin. JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—This paper proves conclusively what many engineers had before only dimly realized, namely, that the inclusion in hydraulic-fill dams of the finer particles of certain clays, introduces an element of danger which should be eliminated either by the exclusion of such minutely fine particles or else by the addition of especially heavy toe fills of rock or other stable material to counteract the semi-liquid pressure of the core.

It appears to be quite generally recognized that the use of a central pool is an essential feature of the process of constructing an earth dam by the hydraulic sluicing method. Of course, a large central pool with a long period of settling is an essential feature if it is considered necessary to deposit from the water all or nearly all the suspended particles. Mr. Hazen, however, has shown that this is undesirable, as it leads to an unstable condition of the core. He advocates a great reduction in the dimensions of the central pool from those in current use, so that the period of settling will be shortened and the finest material not deposited.

The writer, however, raises the question: Why use a central pool at all? The fine impervious material will do the most good at the up-stream face and may be placed there by keeping the sluicing trestles at or near the down-stream face and allowing the soil-laden water to be discharged in an up-stream direction. Fig. 28, which is diagrammatic merely, indicates the various stages in the construction of an earth dam by this modified hydraulic-fill method. The material at the down-stream face is kept heaped up so that no water flows over it. As the water is discharged from the trestle, a great deal of the coarser material will be deposited in the vicinity of the trestle.

The slope is built up so that the finer material, with the water carrying it, will flow up stream. At the up-stream face a low brush dam is maintained, such that there is a small pool near the up-stream face, say, 1 ft. or 2 ft. deep and 15 ft. wide. The surplus water bearing the finest material is allowed to flow over the brush dam and down the up-

* Winston-Salem, N. C.

stream face. If plenty of brush is used, this practice will not cause much trouble from erosion, as the brush protects the face quite effectively. If brush is not available, baffle-boards may be used, and the surplus water and the rejected fines disposed through sluices or pipes. Mr. Justin.

The condition then existing is quite analogous to that illustrated by Mr. Hazen in Fig. 10, where the central pool was contracted to very slight dimensions, resulting in the deposit of material having an effective size of 0.01 mm., which material soon became stable.

With the small pool mentioned, located at the up-stream face, the material deposited would soon become stable. In fact, it would be entirely impossible to use a large pool and deposit a fine material which did not soon become stable, for a brush dam or other slight barrier at the up-stream face would not be strong enough to retain such a semi-liquid fill which would at once break out and flow down the up-stream face.

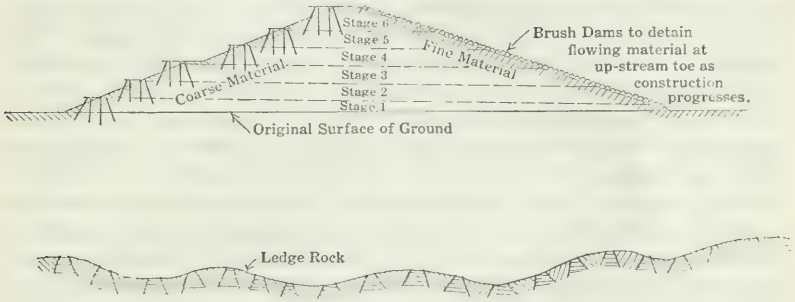


FIG. 28.—METHOD OF CONSTRUCTING EARTH DAM BY SLUICING WITHOUT THE USE OF A CENTRAL POOL.

A modification of this method would be its use in connection with a dam constructed by the semi-hydraulic-fill process. In this method, trains of dump cars are run out on the trestles and dumped. After the cars are dumped, small giants are turned on the dump under the trestle and the material is sluiced away in an up-stream direction, the heavier rocks remaining in the position in which they are dumped from the cars. The remainder of the method is the same as that already outlined.

By excluding the finest particles from the dam, a structure is obtained which, although not theoretically as impervious as if these excessively fine particles were included in the fill, is, nevertheless, practically water-tight. It is claimed for this method that it places the finer material near the up-stream face where it will do the most good, and produces a much more stable structure than one with a puddled core at the center. The down-stream portion of the structure, containing the coarser particles, will act, to a certain extent, as a drain for the

Mr. Justin. more impervious up-stream portion of the dam, and the line of saturation will enter the foundation at a point much farther up stream than would be the case if the usual central pool were used.

By wasting the finest particles a considerable yardage would be sacrificed, but as their inclusion in the structure has been shown to be a source of danger, it is surely the part of true economy to omit them. Also, these rejected fines are not entirely wasted. Flowing up stream from the dam, they will be deposited and tend to fill the interstices of the natural ground surface and thus decrease the flow of water through the soil under the dam.

There are doubtless certain materials, the effective sizes of which are so small that this method would not be applicable but, in that case, it is the part of wisdom to use some method other than hydraulic sluicing for construction.

Mr. Hazen. ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—The writer is gratified to have had nineteen engineers, representing the widest experience in dam construction, take part in the discussion of his paper. The interest thus shown is evidence of the importance of the questions raised, and the writer feels that this broad and representative discussion will do much to clear them up. As these discussions, for the most part, consist of additional data and new points of view not inconsistent with the propositions which the writer has presented, he will not attempt to review them in detail, but will refer only to a few matters.

Some of the discussions show a misconception of the conditions at the Calaveras Dam prior to the slip. Without attempting a complete statement, it may be said that no puddle-core material found an outlet through the dry fill to either face of the Calaveras Dam at any time. The material in the up-stream toe which slipped was placed in horizontal layers only, and not on slopes. The lower toe, which was thinner and which did not slip, was placed in part by dumping on slopes. Hence, so far as the placing of material in this manner tended to weaken the embankment, it would have weakened the lower toe; and as this toe did not fail, certainly the manner of placing played no part in the actual slip.

At the time of the slip the concrete paving on the upper slope of the dam extended only to Elevation 663, while the top of the dam was at Elevation 770. The concrete paving was, therefore, 107 ft. below the top of the embankment and can have had very little influence on the conditions that led to the slip.

The water surface in the lake at the time of the slip was 116 ft. below the top of the dam, and the difference in elevation between the pool in the center of the dam and the lake was certainly sufficient, so that no retardation of drainage can be assumed to have resulted from back pressure.

* New York City.

Mr. Hazen.

It must be remembered also that water had been stored for three seasons before the slip, and that a year previous thereto it had been 20 ft. higher than it was at the time of the slip.

In regard to cost, it may be interesting to note that the company's records show that the actual direct cost of sluicing about 1 500 000 cu. yd. of material to the dam was 26.8 cents per cu. yd. When the costs of camps, roads, engineering, interest during construction, and all other expenses are included, the total cost was 34.6 cents per cu. yd. This, of course, gives no indication of what prices would be in 1920, but the writer believes that neither at that time, nor at the present time, could dry fill be placed at a price comparable with that at which the material was sluiced.

Mr. Holmes' description of the dams for which he has acted as Resident Engineer, with the analyses of materials and tests of permeability, are of the greatest interest. The core has been handled so that the grain size in some cases, on the analyses submitted, is considerably above the limit suggested by the writer; and not only this, but the record of Mr. Holmes' tests on permeability which he has given the writer in greater detail than in the discussion, show that the rate of seepage and the actual leakage through this coarse core material are not greater than those computed by the method used by the writer, but, instead, are somewhat smaller. In other words, there is complete demonstration that core material of this relatively coarse grain size is, for practical purposes, sufficiently water-tight.

Messrs. Burr, Henny, and Binckley refer to the fact that the dividing line between the core and the toe is not vertical, but on an incline, and that there is an upward component of the pressures of the core on the toe, which tends to lift the toe and reduce its resistance to sliding. This matter is more important as the core is wider. It was important at Calaveras. It would not be important for a much narrower core. Mr. Henny contributes an interesting calculation on the reduction in stability growing out of the presence of water back of the dam at various elevations.

Mr. Dunham also suggests that the Calaveras Dam probably slipped on an inclined surface rather than a horizontal one, and the writer believes this surmise is correct. Near the center line of the dam it was clear that the line of slippage was approximately 60 ft. above the bottom of the valley. At the base of the upper toe all the materials slipped (including the gravel from the creek bed that had been placed hydraulically), extending clear down to the natural surface and perhaps below it. These two points are known definitely, otherwise the slipping surface cannot be located, and numerous borings made after the dam slipped did not afford a basis for locating it. The material found in the borings was just the same whether it had slipped or not, and although the borings were made with a view to determining the position

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of the surface of slip, the most careful attention failed to show any indications of it. The surface on which the Calaveras slip occurred, therefore, remains, and must remain, except for the two points mentioned, a matter of conjecture. Mr. Dunham is right, however, in saying that it was not horizontal.

All these matters are of importance in considering the stability of dams. How adequately to take them into account in future plans is the problem that is presented.

Reference has been made to the tests of frictional resistance of the materials at San Pablo. The writer hoped that G. W. Hawley, M. Am. Soc. C. E., Resident Engineer of the San Pablo Dam, who built the apparatus and made the tests, would present a description of them and a statement of the results obtained. As he has not done this, the record of a few results of the first tests made by Mr. Hawley, taken from the writer's notebook, is given in Table 7.

TABLE 7.

Direct pressure, in pounds per square inch.	Approximate depth of fill, in feet, to produce this pressure.	Lateral Pressure, in Terms of Direct Pressure Required to Displace Moist Material.		
		Orindian blue shale.	Monterey brown shale.	Worn brown shale, sand and small stones from dam.
17.4	21	0.67	0.78	0.57
34.9	42	0.54	0.67	0.54
52.4	63	0.46	0.61	0.55
69.8	84	0.45	0.55	0.57
87.3	105	0.45	0.53	0.56
104.7	126	0.45	0.50	0.56
122.2	147	0.45	0.50	0.56
139.6	168	0.45	0.49	0.50
157.1	189	0.44	0.48	0.54
174.6	210	0.45	0.46	0.54
Approximate average for results with pressures corresponding to more than 100-ft. depth of fill		0.45	0.49	0.54

Each figure in Table 7 represents the mean between two observed values. The first was of the pressure required to start the movement; that is, the maximum pressure observed before there was a slip; and the second was the pressure required to keep the slip moving after it was once started. In a general way, the latter pressure was about 10% less than the former. In other words, the friction required to start movement was 5% greater than that indicated by the figures, while the friction after movement was 5% less.

The writer believes that this kind of experimentation offers promise of better knowledge of different materials, and of the effect of varying

quantities of water in the same material; and he hopes that a complete record of the San Pablo tests may be published, and tests of other materials made. Mr. Hazen.

Answering Mr. Dunham's questions, the section of the dam for which he asks may be found in *Engineering News-Record,* of December 26th, 1918, and a summary of the data relating to the slip may be found there and also in the issue of April 4th, 1918.

Mr. Morgan thinks that a clear distinction should be made between the compacting of the material by drainage and that by consolidation through settling under water. It seems to the writer that such a distinction may be hardly practicable, and he is inclined to include the settling through water (which necessarily involves an upward flow of the water through the material) under the term drainage.

The little volcanoes at the top of soft core material, which Mr. Morgan has mentioned, have been observed by the writer, who believes that they probably occur only in material with a large percentage of voids, and that they do contribute largely to the rapid first consolidation of material. Later, the material becomes more solid, and this volcano action stops. It must stop before the material has acquired any useful degree of stability, and for this reason the volcano action cannot be an element of importance in the final consolidation. It may be that it accounts for the original consolidation of the material to approximately 50% of voids in the course of a few months, which would otherwise be difficult to explain.

There can be no doubt that drainage of saturated material is essential to consolidation. The core material is deposited under water. Its voids are completely filled with water. Consolidation can take place only as the surplus water is excluded.

The conditions of flow are essentially the same whether the movement of the water is vertical or horizontal. It may be that the hydraulic slope for the vertical flow will be greater than was mentioned in the paper. This proposed value did refer to horizontal flow. For vertical flow the hydraulic slope may be taken as the specific gravity of the material less 1.00. A material with 50% of voids has a specific gravity wet of 1.825, and the hydraulic slope for upward flow is therefore 0.825. Using an estimated rate of seepage based on this value would change the figure, but not the essential conditions as described.

How to get the excess water out of the core material is the one dominant problem with fine-grained core material. It is the water that holds the particles apart and prevents consolidation; if it can be taken out by drainage, consolidation follows as a matter of course and to the full extent that water is removed.

In considering experience with shallower materials elsewhere, evaporation from the surface and bringing water up from below by capillarity is often a dominant factor, and this must be kept in mind in making comparisons with observations obtained under such conditions.

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Mr. Paul thinks that the writer's descriptions of 50% material are inconsistent in that at one time it is described as being capable of standing up in a boring driven beyond the casing for several feet, and in another paragraph as exerting full lateral pressure. The explanation of the apparent discrepancy lies in the cohesion of the material. Cohesion is real and important. It is cohesion that holds the vertical sides of a cut in such material to a depth of several feet. It is cohesion that gives it the firmness of stiff putty. It is cohesion that holds the particles together as an arch about the borings below the end of the casing. Cohesion exists equally in a great mass of material under heavy pressure, but there it becomes so small in comparison with the pressure, that it is no longer able to maintain stability and the material becomes capable of flowing.

The depths of penetration mentioned by Mr. Paul, as observed at Calaveras, are understood by the writer to be maximum depths at special points at a time long before the slip. Before the slip occurred there had been a temporary cessation of operations, and the core material had consolidated and hardened to a point where penetration was resisted at a depth of a very few feet; and so far as this kind of evidence goes, the core material at the Calaveras Dam at the time of the slip was satisfactory.

The one great lesson to be learned from the Calaveras slip is that material consolidated to this extent and having the consistency of stiff putty, capable of resisting penetration is, nevertheless, capable of flowing in large masses.

Another question in which the same line of thought is involved is whether deductions from readings of small and very ingenious pressure gauges may be safely used in calculations of pressure in large masses. The writer is in doubt on this question, and is disposed to proceed slowly pending further demonstration. The question which he raises is not as to the accuracy of the record of the gauges (of which he has no knowledge), but whether, in view of cohesion, actual pressures on very small areas really indicate conditions in large masses. For instance, would the pressure gauge against the surface of material standing beyond the casing of a bore hole show full lateral pressure? The writer is in no doubt of the fact that some material that will stand in that way without support is, nevertheless, capable of flowing almost like water when the mass is sufficiently great. Mr. Goldbeck's gauges seem to be ingenious devices, and the results obtained by their use in actual dams will be studied with interest.

Mr. Muckleston's observations suggest the possibility of chemical treatment to improve the physical condition of colloidal clay. This certainly opens interesting possibilities.

Mr. O'Shaughnessy suggests reducing the flow line of the Calaveras Reservoir 37 ft., in the interest of greater safety. Reducing the flow line by this amount would reduce the capacity of the reservoir by about

19 300 000 000 gal., or by about 40 per cent. The full proposed capacity is not too much to hold the waters of floods that occur only at intervals of several years, and make them available at all times, especially during the dry years sometimes experienced in California. If the flow line should be lowered as proposed, it would reduce the maintainable supply through a series of years by 10 000 000 gal. per day. In the San Francisco District, 10 000 000 gal. of water per day are worth a certain number of millions of dollars. To reduce the height of the dam, and with it the capacity and value of the source, should not be done unless there is valid reason for believing that a dam built to the full height cannot be made safe. Mr. Hazen.

Mr. Binckley's statement in regard to Mr. Schuyler's later views and experience, unfortunately not expressed in writing before Mr. Schuyler's death, are most interesting and helpful; and it is gratifying to the writer that the views which he has presented are supported in this manner.

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Paper No. 1459

NOTES ON TUNNEL LINING FOR SOFT GROUND*

By S. Johannesson and B. H. M. Hewett, Members, Am. Soc. C. E.

With Discussion by Messrs. J. C. Meem, E. G. Haines, Philip Aylett, Walter C. Parmley, C. M. Holland, T. Kennard Thomson, Orrin L. Brodie, and S. Johannesson and B. H. M. Hewett.

Synopsis.

In the construction of tunnels, particularly under compressed air in soft soil, the engineer has met a number of difficulties connected with the lining, often obscure in their origin, which for that reason have caused anxiety, delay, and unforeseen expense.

In this paper the more important of these difficulties are stated and methods of avoiding them suggested. These suggestions are discussed both from a practical and theoretical point of view. It is the purpose first to attempt to determine the causes for the distortions and displacements common to tunnel linings, and, second, to suggest how they may be avoided by the modification of existing practice. The subject, however, is so extensive that, in order to keep the paper within reasonable limits, a general outline only will be presented.

For the sake of convenience, the paper is divided into sections as follows:

1.—Behavior of tunnel lining during construction.

2.—Determination of stresses in tunnel linings.

3.—Conditions for stable equilibrium.

4.—Stresses in tunnel lining during construction.

* Presented at the meeting of April 21st, 1920.

5.—North River silt.
6.—Ruling requirements for the erection and design of tunnel lining.
7.—Design of tunnel lining.
8.—Materials of construction.
9.—Secondary lining.
10.—Cost of tunnel linings.
11.—Conclusions.

1.—Behavior of Tunnel Lining During Construction.

A common experience when building a shield-driven cast-iron lined tunnel is that, although it may be possible to erect the lining true to line, level, and shape, it is difficult and even impossible to keep it so during the construction period.

Immediately after the shield has left a newly erected ring of the lining the ring may move up or down and as likely as not it will change its shape, sometimes in one direction and sometimes in another. If the deformation threatens to become excessive, the tunnel has to be braced. Afterward it may be found that this bracing, although carefully placed, becomes loose and ineffective because the lining has started to deform in another direction. Then the bolts may have to be retightened and the water-proofing of the joints and the bolt holes may have to be done again every time the lining changes its shape.

The up or down movements may be even more serious. In the Pennsylvania Railroad Tunnels under the North River at New York City at one time the tunnel started to rise suddenly immediately behind the shield for some at that time unexplained reason, and rose as much as 12 in. above the level at which it was erected. This naturally caused much concern. The work was closed down for about one month and the tunnel lining redesigned.

In another tunnel under the East River at New York City, the lining settled at one place to such an extent that there was not sufficient clearance left for the traffic to pass through, and thus necessitated the reconstruction of this part of the tunnel.

It is evident that to take care of these distortions and displacements is expensive both in money and time and adds greatly to the cost of the finished structure. Therefore they should, if possible, be avoided.

2.—Determination of Stresses in Tunnel Linings.

The primary function of the lining of a tunnel is to support the surrounding soil. In addition, it is often necessary and nearly always desirable that the lining should be capable of preventing the infiltration of water which may be present in the ground under a consirable head. Furthermore, when the tunnel is constructed by the shield method the lining must be able to support the thrust of the shield rams.

The lining of a tunnel is subject to stresses caused by the external forces, and only when these are known is it possible to determine the stresses in the lining.

Assuming material with small angle of repose, the vertical forces on the upper half of the tunnel are the weight of the soil, the weight of the lining, and the possible live loads above the tunnel within two vertical planes tangent to the sides of the tunnel. The force on any given area of the tunnel is that portion of the forces which is vertically above the area.

The vertical forces on the lower half of the tunnel are equal to the vertical forces on the upper half of the lining, plus the weight of the tunnel and its contents. The distribution of these forces is uncertain, but can generally be assumed with sufficient accuracy. The horizontal forces, which must be balanced on the two sides of the tunnel, as the tunnel otherwise will be in sidewise motion, can be determined by means of Rankine's "Theory of Conjugate Stresses."

Let p = the vertical load on a unit of horizontal surface within the ground;

p_1 = the horizontal soil pressure on a unit of vertical surface at the same depth; and

α = the angle of repose of the soil.

Then, in accordance with Rankine's theory,

$$p_1 = p \frac{1 - \sin. \alpha}{1 + \sin. \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This formula represents the active horizontal pressure of the soil. The soil is, however, able to resist a greater horizontal force, represented by

$$p_2 = p \frac{1 + \sin. \alpha}{1 - \sin. \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

This force, p_2, is called the passive pressure of the soil.

The horizontal pressure on the tunnel is always at least equal to the active pressure, p_1, but if the condition of the internal stresses in the tunnel lining should so require, a greater horizontal pressure will be developed of such a magnitude as will be required to provide the necessary equilibrium, but under no circumstances greater than p_2.

In order to find the stresses in the tunnel lining it is necessary to determine which value of the horizontal force, between the limits of p_1 and p_2, is in action. This can be determined by the fact that such a horizontal force is in action as will produce the least internal work.

The least internal work will be produced if the stress polygon coincides with the neutral axis of the tunnel lining. To find if this coincidence is possible, it is necessary, first, to determine whether a horizontal force which would produce the coincidence might be applied, and, second, whether such a horizontal force would come within the limits of the active and passive forces. If it does, the stress polygon will coincide with the neutral axis, and the tunnel lining will be in direct compression only.

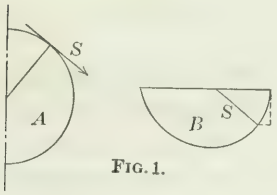


FIG. 1.

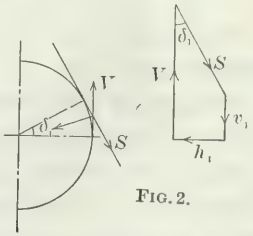


FIG. 2.

In Fig. 1, A represents half the circumference of a tunnel, and B a force diagram for the tunnel. It will be seen that, if the direction of the force S is parallel to the tangent to the corresponding point of the circumference, the stress polygon will follow the neutral axis; it will be seen, also, that for any given vertical force there is always a horizontal force which will produce the desired results. The value of this horizontal force can be determined as follows:

The forces acting on the portion of the lining between the horizontal radius and a radius forming an angle δ_1 with the horizontal are the tangential stress, S, in the lining, the vertical stress, V, and the external forces with a vertical component v_1 and a horizontal component h_1.

Fig. 2 shows that

$$h_1 = (V - v_1) \tan. \delta_1 \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where V is the vertical load on one-half of the tunnel, and v_1 is the vertical load on the portion of the lining considered.

The horizontal force on a portion of the lining between two radii forming angles of δ_1 and δ_2, respectively, with the horizontal is

$$h_1 - h_2 = (V - v_1) \tan. \delta_1 - (V - v_2) \tan. \delta_2 \ldots\ldots\ldots\ldots (4)$$

In a circular tunnel with a radius, r, the horizontal pressure per unit of area is

$$p_i = \frac{h_1 - h_2}{r (\sin. \delta_2 - \sin. \delta_1)}$$

or

$$p_i = \frac{(V - v_1) \tan. \delta_1 - (V - v_2) \tan. \delta_2}{r (\sin. \delta_2 - \sin. \delta_1)} \ldots\ldots\ldots (5)$$

This pressure, p_i, is, in the following, termed the ideal pressure.

In a more convenient form for use, p_i may be expressed by the following equation, which gives an approximate value only and in which w represents the unit weight of the soil, and δ the radial angle:

$$p_i = p_1 + 0.11 (2 + \sin. \delta) (1 - 2 \sin. \delta) w r \ldots\ldots\ldots (6)$$

If $p_1 < p_i < p_2$, the tunnel lining will be subject to direct compression stresses only.

Assuming that the vertical forces on the lower half of the tunnel are distributed in the same way as they are on the upper half, which is near enough for the present purpose, in order to obtain the condition that the lining may be in direct compression only the active horizontal pressure at the bottom of the tunnel should be no larger than the ideal pressure, p_i, at the top of the tunnel.

In order to determine what angles of repose would produce such pressures under certain given conditions, the pressures, p_i, have been determined for circular tunnels with diameters of 12, 18, 24, 30 and 36 ft., respectively, and with the assumption that the vertical pressure at the top of the tunnel is 4 000 lb. per sq. ft. and that the weight of the material is 120 lb. per cu. ft. The results are plotted on Fig. 3, and by applying Equation (1) it is found that, under the assumptions made, direct compressive stresses only will occur when the angle of repose is not less than the values given in the following table:

Diameter of tunnel, in feet.	12	18	24	30	36
Angle of repose.....	10° 30′	14° 30′	18° 30′	22° 0′	25° 30′

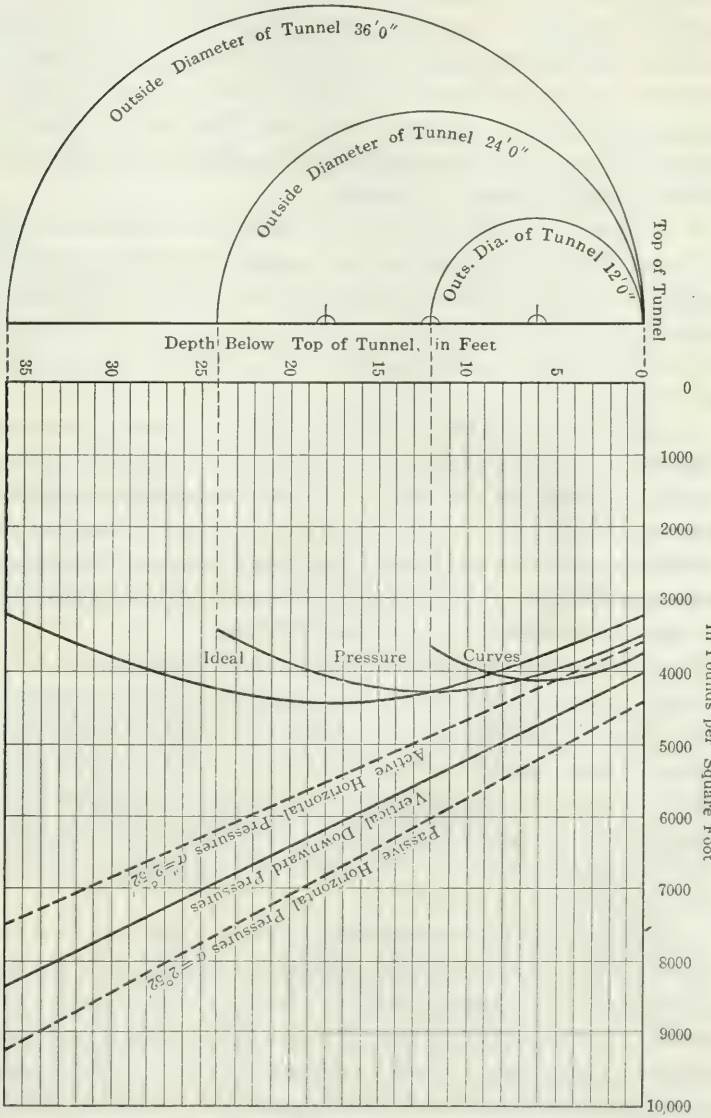

Outside Diameter of Tunnel 36′0″
Outside Diameter of Tunnel 24′0″
Outs. Dia. of Tunnel 12′0″
Top of Tunnel
Depth Below Top of Tunnel, in Feet
35
30
25
20
15
10
5
0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10,000
Ideal
Pressure
Curves
Active Horizontal Pressures
Vertical Downward Pressures
Passive Horizontal Pressures
Vertical, Horizontal and Ideal Horizontal Earth Pressures, In Pounds per Square Foot

FIG. 3.

If the vertical pressure at the top of the tunnel is less, the angles will be greater, and if the weight per cubic foot of the material is less, the angles will be less; but generally it may be said, that in order to obtain direct compressive stresses only in the lining of a circular tunnel, the diameter of the tunnel must be limited to a certain value which varies directly as the value of the angle of repose. This limiting value of the diameter is about 12 ft. when the angle of repose is 10°, and about 43 ft. when the angle of repose is 30 degrees.

If the angle of repose of the soil is less than as determined above, the lining will be subject to bending stresses in addition to the direct compression stresses. These bending stresses may be determined by means of a stress diagram. It is evident that the horizontal forces to apply are the active horizontal pressures, when they are larger than the ideal pressures.

In Equation (5) the term, r (sin. δ_2 — sin. δ_1), represents the vertical projection of the area of the tunnel lining upon which the horizontal pressure is acting. If this area is reduced without similarly reducing the horizontal projection of the area, the value of p_i will increase. Such a reduction can be accomplished by making the tunnel elliptical, with the major axis horizontal. The ratio between the horizontal and vertical diameters of the ellipse can be found as follows:

First, determine the maximum passive horizontal pressure per unit of area at the top of the tunnel, and the active horizontal pressure per unit of area at the bottom. If the latter is equal to the former, an elliptical tunnel may be constructed which will be subject to compressive stresses only, and the ratio between the horizontal and vertical diameters of the ellipse will be equal to the square root of the ratio between the active pressure at the bottom of the tunnel and the ideal pressure at the top of the tunnel.

If the maximum passive horizontal pressure at the top of the tunnel is less than the active horizontal pressure at the bottom of the tunnel the lining will be subject to bending stresses which, however, will be as small as possible if the ratio between the horizontal and vertical diameters of the ellipse is equal to the square root of the ratio between the maximum passive pressure at the top of the tunnel and the corresponding ideal pressure.

If the maximum passive horizontal pressure at the top of the tunnel is greater than the active horizontal pressure at the bottom of the tun-

nel, elliptical tunnels subject to direct compressive stresses only may be designed, with the ratio between the horizontal and vertical diameters varying between the square root of the ratio which the active bottom horizontal pressure bears to the top ideal pressure and the square root of the ratio which the maximum passive top pressure bears to the top ideal pressure.

The ratio between the vertical and the horizontal diameter in soil with an angle of repose of 2° 52′, weighing 120 lb. per cu. ft. and when the pressure at the top of the tunnel is 4 000 lb., is approximately as 9 to 10.

It may be stated, therefore, that in soil with an angle of repose producing bending stresses in a circular tunnel, it is possible to reduce and sometimes to eliminate the bending stress by making the shape of the tunnel elliptical.

3.—Conditions for Stable Equilibrium.

If the angle of repose is small, the distribution of the vertical forces on the lower half of the tunnel may be different from that assumed, because the active vertical forces become effective. In that case the distribution may be assumed to be as follows: Determine the upward forces which, in accordance with Rankine's theory are, per unit of area:

$$p_3 = p \left(\frac{1 - \sin. \alpha}{1 + \sin. \alpha} \right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

and distribute the difference between the upward and downward forces, including the weight of the tunnel, uniformly over the horizontal diameter.

If this difference is less than the weight of the tunnel, passive upward vertical forces are developed, and according to Rankine the maximum value of these is:

$$p_4 = p \left(\frac{1 + \sin. \alpha}{1 - \sin. \alpha} \right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

It may be said, therefore, that if the total vertical downward forces are less than or equal to the total maximum passive upward forces as determined by Equation (8), the tunnel will not sink.

If the total active upward forces as determined by Equation (7) are greater than the total downward forces, it does not necessarily follow that the tunnel will rise, because the resistance to upward motion is greater than the total downward load; for, in addition to overcoming

the resistance of the load, the frictional resistance on two vertical planes from the top of the soil to the bottom of the tunnel has to be overcome.

Let D = the distance from the top of the soil to the bottom of the tunnel;

q_1 = the unit horizontal pressure at the top of soil; and

q_2 = the unit horizontal pressure at the bottom of the tunnel.

Then the frictional resistance will be

$$F = (q_1 + q_2)\ D \text{ tan. } \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

It follows, therefore, that if the sum of the total downward load and the frictional resistance is greater than the total active upward force, the tunnel will not rise.

4.—Stresses in Tunnel Lining During Construction.

During the period of construction, the lining is subject to external forces and to conditions different from those which exist when this period of disturbance is past. In order to secure the safety of the tunnel during this period and to carry out the work of construction economically, it is necessary to determine in advance the stresses to which the lining may be subjected during the construction. This is particularly true when the tunnel is driven by the shield method through water-bearing ground.

Since the tunnel lining is used as the abutment for the shield rams, the stresses in the lining are influenced by the pressures exerted on it by the rams. These ram stresses consist of direct compression parallel to the axis of the tunnel, when the line of pressure coincides with the neutral axis of the skin of the lining. If not, additional weight of lining material is needed to take up stresses which are wholly temporary and constructional and which do not exist after the driving period is over. It is ordinarily present, however, with the usual type of cast-iron lining, and has been known to cause breakage of the lining and damage to the rams, particularly in tunnels of large diameter.

The conditions under which the shield is driven forward vary with the character of the ground. If the soil is firm it is excavated ahead of the shield, and the duty of the shield rams is mainly to overcome the frictional resistance. When the ground is soft and has a small angle of repose the shield may be used to displace part or all of the soil to be

removed to make room for the tunnel. In this case not only frictional resistance, but also the resistance to displacement has to be overcome.

When the shield is driven forward, displacing the soil, and particularly when this is done with the doors closed, the active horizontal pressures directly in front of the shield are increased to the value of the passive pressures, and at the same time rupture of the body of the soil occurs. In North River silt this increase is about 20 per cent. A proportional increase of the active upward forces will necessarily occur. If the existing condition of stable equilibrium is such that the total downward load and the frictional resistance about balance the upward forces, such an increase of the upward forces may cause the shield to have a strong tendency to "climb", and the tunnel to rise immediately after the shield has left the lining. This tendency to rise will extend only some distance behind the shield and will disappear when the abnormal active ground pressures have had time to readjust themselves. It may be relieved by admitting sufficient quantities of soil through the shield doors.

At the moment the tail of the shield leaves the lining, when the shield is pushed ahead, there exists an empty annular space around the lining. In firm soil it is probable that the first result will be that the lining will drop down to rest on the ground surface at the bottom and that the soil above the tunnel will close down until it bears against the upper half of the tunnel. Later, probably, the ground will close in at the sides. It is evident that the effect will be to introduce a bending stress in the lining, so that if the lining is not able to carry such stress the top of the tunnel will be pushed down and the sides will spread out until they bear against the soil. This tendency to distortion may be decreased, however, by filling the space with grout or other solid material.

In ground with a small angle of repose it is possible that, on account of its greater mobility, the soil may close around the tunnel immediately after the shield is pushed ahead. The construction conditions, however, will have appreciable influence on the stresses in the lining.

To begin with, the pressure of the compressed air produces bending stresses in the lining.

To illustrate this stress, diagrams have been made to show the varying conditions of stress produced by varying conditions of air pressure. Fig. 4 shows a force polygon and stress polygon under certain given conditions when the tunnel is under normal air pressure. Fig. 5 shows

the force polygon and stress polygon for the same tunnel under an air pressure of 25 lb. and Fig. 6 a force polygon and stress polygon when the air pressure balances the external pressure at the crown of the tunnel.

A comparison of these diagrams shows that while the internal stresses in the lining decrease as the air pressure increases, the eccentricity of the stress increases with increased air pressure. In cast-iron lining of the usual type, the bending moment introduced by the air

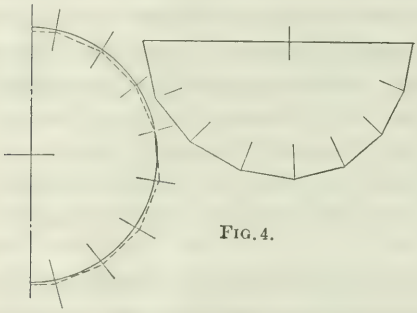


FIG. 4.

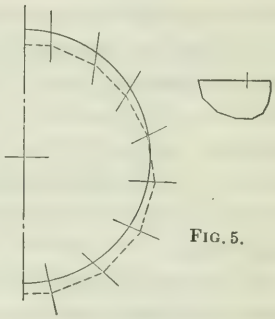


FIG. 5.

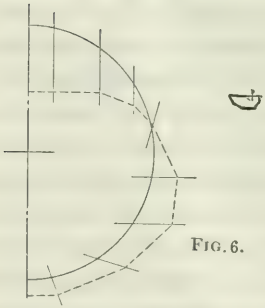


FIG. 6.

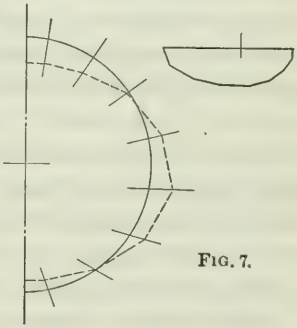


FIG. 7.

pressure may cause a tensile stress of several thousand pounds per square inch. As the cast-iron lining is not a continuous elastic structure, the effect of these bending stresses will be to open the joints between the segments. An inspection of the diagrams will show that the tendency will be to open the joints at the bottom of the tunnel on the inside and at the top of the tunnel on the outside. However, as the soil at the top may resist the tendency of upward displacement the probable net result will be a shortening of the vertical and a corresponding lengthening of the horizontal diameter.

As already stated, the pushing of the shield ahead without taking in the displaced soil increases the earth pressures in the vicinity of the shield. Similar results may be caused by pushing forward the shield in an adjacent tunnel, if that tunnel is sufficiently close. Fig. 7 shows the stress curve under these conditions. The tendency to distortion in this case is to increase the vertical and to decrease the horizontal diameter.

Stating them in the proper order, the tendencies to distortion (and if the lining is not able to withstand the stress, the actual distortions) will be, in soil with a small angle of repose, as follows: First, immediately after leaving the shield, the lining will rise, if the conditions are present, and at the same time increase the length of its vertical diameter and decrease that of the horizontal diameter. Soon after, when the soil conditions have become normal, the tendency will be to push in the lower half of the lining, thereby shortening the vertical diameter and lengthening the horizontal diameter. This tendency becomes more aggravated with an increase of air pressure. Finally, when the air pressure is removed, the tendency to distortion disappears, to a great extent, but at the same time the magnitude of the direct stress is increased.

The conclusions here arrived at correspond closely to the actual behavior of the tunnels driven through North River silt. The excessive rise of the Pennsylvania North River Tunnel previously mentioned, which caused the work to be stopped for a time, happened when both the tunnels were being driven close to one another and with the doors closed. The rise was effectively checked by driving one shield well in advance of the other and by taking a sufficiently large quantity of the displaced soil into the tunnel through the shield doors; in fact, it was found that the path of the shield could be adjusted readily by gauging the volume of the soil taken into the tunnel. The volume naturally depends upon a number of conditions, including the diameter of the tunnel; if that is small enough, it may be possible to push the shield ahead with the doors closed without producing a rise of the tunnel; in fact, this was frequently the experience in the tunnels of the Hudson and Manhattan Railroad under the North River which, in addition, were so far apart that the adjacent shields had little or no effect upon each other.

The flattening tendency indicated by the theoretical determination of stresses under air pressure, was a common experience in all tunnels

driven through the North River silt. In all cases, the ring of the lining was originally built true to shape. If after erection it began immediately to increase its vertical diameter, this change of shape stopped within about two weeks; thereafter the lining commenced to increase its horizontal diameter and continued to do so until the secondary lining was placed. If no initial shortening of the horizontal diameter occurred, the lining began, soon after erection, to flatten and this often continued until checked by internal bracing.

The conclusions to be drawn from these conditions are that, for safety and economy, it may be advisable to design the lining so as to be able to carry not only the permanent stresses, but also the construction stresses, which include bending, even though the permanent forces produce compressive stresses only, and that consequently the primary lining should be the entire carrying member.

5.—North River Silt.

In the foregoing discussion, references have been made to soils with a small angle of repose, and it may be pertinent, therefore, to state some of the characteristics of the North River silt underlying the North River and adjacent waters, which is an example of a soil with a small angle of repose.

During the period of construction of the tunnels under the North River for the Pennsylvania Railroad and the Hudson and Manhattan Railroad, a great number of tests, experiments, and analyses were made to determine the characteristics of the silt. As a number of tunnels already have been built in this material and more will be built, the most important results obtained will be stated here.

The weight of the silt increases with the depth below the river bed, at least to the depth that tests were made, which was to the bottom of the tunnels. At a depth of 15 ft. below the river bed, the silt weighs about 104 lb. per cu. ft.; at a depth of 38 ft. it weighs about 110 lb. and the average weight of the silt surrounding the tunnels is 108 lb. per cu. ft. The silt surrounding the tunnels contains 55% of water by volume.

The unit weight of the silt and its water content vary with the tide. At high tide the weight is greater than at low tide and the water content is less. This points to the probability that the solid particles of the silt come closer together at high than at low tide, and this is also evidenced by the fact that there is a reversing tidal motion of the silt, which rises as the tide falls and falls as the tide rises. This tidal rise and fall

amounts to an average of about $\frac{1}{8}$ in. at the level of the tunnels. It is to be presumed that it varies from zero at the bottom of the silt to a maximum at the top.

A body embedded in the silt will move up and down with the silt, but if it is in stable equilibrium, it will not move in relation to the silt.

The angle of repose of the silt surrounding the tunnels was determined to be less than 3 degrees. This was done by analyzing many hundreds of observations of the external pressures on the tunnel lining, obtained by means of specially constructed and highly sensitive pressure gauges inserted in holes in the skin of the cast-iron lining.

The fact that a shield with its doors completely closed can be forced bodily through the silt without taking a shovelful of muck into the tunnel and without requiring undue jack pressures to be developed shows that this particular soil is essentially a viscous fluid. Such a thing could not be done in clay or sand or in a mixture of such. Observations on the jack pressures used to start an actual shield in movement through the silt confirm, when analyzed, the fact that the material has an angle of repose of less than 3 degrees.

Another striking indication of the quasi-fluid nature of the silt is afforded by its vertical fluctuation due to the tidal rise and fall of the water in the North River above it. The ordinary range of tide here is only 4 or $4\frac{1}{2}$ ft. and yet, at the elevation at which the tunnels lie, this small tidal range produces a change of elevation in the silt of about $\frac{1}{8}$ in. Under a tidal range of 10 ft., a change of $\frac{1}{4}$ in. in the elevation of the silt, or rather in that of the tunnel embedded in the silt, has been observed.

Certain other characteristics of the silt, observed during the construction of tunnels through this soil, should also be noted. When subjected to air pressure, the silt has the nature of a stiff clay and its consistency may readily be varied by changing the air pressure, that is to say, when the air pressure nearly balances the soil pressure, the material becomes quite stiff, but as the air pressure is reduced the silt becomes softer and softer.

6.—Ruling Requirements for the Erection and Design of Tunnel Lining.

It is evident from the previous discussion that, if the tunnel lining is designed in such a manner that it is able to carry the stresses to which it may be subjected, both during the period of construction and there-

after, and if the work of construction is carried out properly and with due regard to the peculiarities of the soil through which the tunnel is driven, the usual difficulties and expense experienced in keeping the tunnel true to line, grade, and shape will be avoided.

The remedies for the rising or sinking of the tunnel are apparent, providing the tunnel is in stable equilibrium, and this is naturally a primary necessity. When the tunnel rises, it is probably due to the existence of excessive external pressures caused either by an adjacent tunnel shield or by the displacement of the soil by the shield of the tunnel itself.

The prevention of rising is to make such an adjustment of the quantity of material brought in through the shield doors that these excessive external pressures are not brought into existence. When the tunnel sinks it is plainly due to leakage of water and soil through the joints of the lining over the lower portion of the periphery. The prevention consists in making the lining water and mud tight at the time of its initial erection, or as soon after as possible.

The prevention of deformation lies entirely in the design. If the lining is designed so that it is able to carry the stresses that may be brought upon it, both during and after the construction period, no deformation will occur. It is of importance, therefore, that the lining should be designed with due regard to this requirement.

When the art of building cast-iron lined subaqueous tunnels was in its infancy the size of the tunnels was kept within small limits and the tunnels were built through ground that was favorable for safe construction.

As time went on, and there developed the necessity for tunnels of larger size and in fixed locations regardless of the suitability of the soil and depth of waterway encountered, fresh problems were presented to the tunnel engineer, which indicated that either the design or the material of the lining, or both, had outgrown their original usefulness from the standpoint of safety as well as of economy. In what follows, therefore, the design of tunnel linings will be discussed while keeping in mind the safety and the economy of the design under varying conditions, both of size and surrounding material.

7.—Design of Tunnel Lining.

In undertaking the design of a tunnel lining for any given purpose and under any given conditions, the following points have to be kept

in mind: The shape, the strength, the material of construction, the water-proofing, the time required for construction, and the cost of maintenance and operation.

As regards the shape, if the cross-section of the tunnel is such that the lining is subject to direct compressive stresses only or, if that is not obtainable, to the least possible bending stresses, the least quantity of construction material will be required per unit length of the periphery. As stated in the section on determination of stresses, this points to the use of an elliptical cross-section, at least when the angle of repose of the soil is small. When it is possible to arrange the internal clearance profile in an elliptical cross-section so that no space is wasted, a further economy is obtained by having decreased the vertical diameter, because both the stresses in the lining and the difficulties of construction are thereby decreased. The possibilities of making the shape elliptical have to be studied in each specific case, and therefore no detailed comparisons, as regard economy, between circular and elliptical tunnels can be attempted in this paper.

In most cases it is necessary or advisable to maintain the proper lines and grades, either to permit the passage of the traffic for which the tunnel is intended or for the sake of appearance. Therefore, some additional clearance, say, from 3 to 6 in. all around the inner periphery, should be provided for the adjustment of the unavoidable deviations from the proper lines and grades that will occur during the driving of the tunnel.

As regards the strength of the lining, it has been shown that, under certain conditions, the lining is subject to bending stresses. For the purpose of avoiding non-elastic deformation, the lining should be able to carry such stresses. The ability of various lining materials to do this will be discussed in the following section. However, it may be repeated here that on account of the fact that the bending stresses occur during the construction period, as well as after, the lining placed at the time of construction should be strong enough to carry all stresses without any secondary lining placed at any later date.

The economy of construction depends not only on the unit cost of the material or materials of which the lining is constructed, but also on the quantity of material required. This includes not only the cost of procuring the materials, but also their cost of erection. It will be remembered that, if the tunnel is driven under air pressure, the cost of

labor under these conditions is exceptionally high, and that, therefore, such materials should be selected, and so arranged, as to require as little labor under compressed air as possible.

In this connection, it may be noted that while the internal diameter of the tunnel is determined by the use for which the tunnel is intended, the external diameter and consequently the quantity of excavation is in addition determined by the thickness of the lining. Therefore, in comparing various types of lining from the point of economy, the quantity of excavation should also be considered.

A further matter of economy is that of the period of time required to finish the tunnel ready for operation. Generally it is desired to complete the construction as soon as possible, because the tunnel is imperatively needed. The loss of money by reason of the tunnel not being ready for operation often cannot be estimated definitely but is, nevertheless, an important item. In addition, the monthly overhead expenses continue about the same during the whole period of construction, and the interest on the capital invested increases with the time of construction. This indicates as advisable an investigation of the relative periods of time needed to construct tunnels of various sizes, shapes, and materials.

A preliminary study of the rates of progress attained in the different New York river tunnels showed that, generally speaking, the larger the tunnel the less progress, measured in linear feet of tunnel driven per month, was attained under any given set of conditions.

When the weight of cast-iron lining corresponding to the linear feet of tunnel driven per month was computed, it was found that there was a fairly constant weight of cast-iron lining erected per month under any given set of ground conditions, irrespective of the size of the tunnel. This is shown in Table 1.

TABLE 1.—Weight of Cast-Iron Lining Erected per Month in Compressed Air Tunnels at New York City.

Material.	Weight of Cast-Iron Lining Erected per Month, in Tons.			
	P. R. R., North River.	H. & M. R. R., North River.	Dual Subway, East River.	Average.
Sand, gravel, etc.	660	.....	484	550
Mixed ground	300	300	182	250
Rock	235	220	243	230
Silt	1 650	1 800	1 665	1 750

The cast-iron lining of the tunnels built in the neighborhood of New York City weighs, in pounds per linear foot of tunnel, about twenty-two times the square of the outside diameter expressed in feet. Consequently, the fact that the weight of iron lining erected per month under any given conditions of ground is more or less constant, makes it possible to foretell the approximate monthly progress which a tunnel of any given diameter will make under any given conditions of ground. This estimate has been plotted to show the result in graphical form (Fig. 8), and it is interesting to see that the line of progress starts at zero for zero diameter, rises rapidly to a maximum for a tunnel of about 10 ft. in diameter, and then diminishes as the diameter of the tunnel is increased.

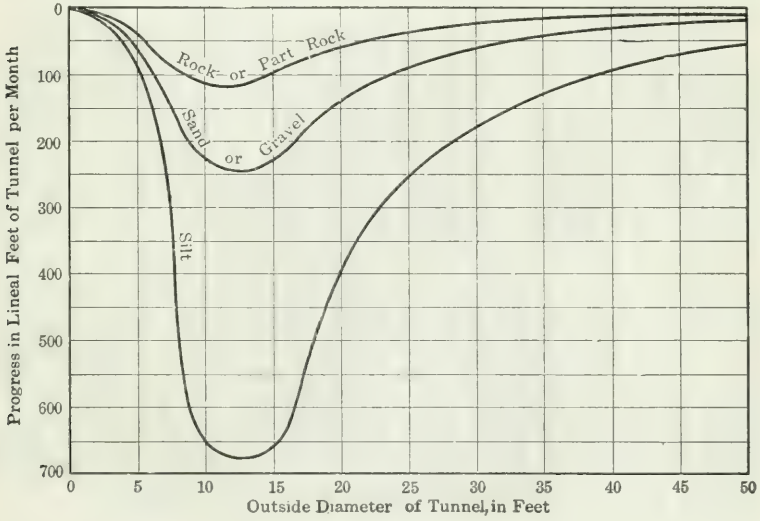


FIG. 8.

It may be said, therefore, that for shield-driven tunnels lined with cast iron of the usual type (and probably for shield-driven tunnels lined with any kind of material) the smaller the diameter, down to about 10 ft., the quicker progress will be made, the less the overhead and the interest charges during construction will be, and the sooner the tunnel will be ready for operation. This means that it is economically sound (provided that the purpose for which the tunnel is intended is not impaired thereby) to build a number of small tunnels instead of one large one. For example, it is thoroughly sound to provide a two-line railway tunnel by building two single-line tunnels rather than one

double-line tunnel, provided the quantity of material and work for the two single tunnels does not become so large in comparison with those items for the double-line tunnel that the advantage gained in the shorter construction period is counterbalanced.

The same reasoning applies to sewer tunnels, water supply tunnels and in fact to all cases where the traffic may be subdivided. A further incidental advantage is hereby gained, in that with a plurality of tunnels one or more may be shut down for repairs at times of slack traffic without shutting off the entire traffic. A constructional advantage gained by use of several small tunnels in place of one large one is that the difference in pressure between the top and bottom of the tunnel is less.

Most tunnels which have to be driven under navigable waters are limited as to the depth below the water surface at which they may be placed. For example, public authority may require that no encroachment may be made on a water depth of 40 ft. This fixes the elevation of the crown of the tunnel and this in turn fixes the position of the roadway or track which is placed within the tunnel. The smaller the vertical diameter of the tunnel the nearer to the surface this roadway or track may be kept and thus not only are the approaches made shorter or the gradients easier, and the total length of the tunnel decreased, but the vertical height through which every person or vehicle which uses the tunnel has first to be lowered and then to be raised is diminished. Thus a saving in energy is introduced and this saving is continuous throughout the entire life of the structure. If capitalized, this saving may amount to an appreciable sum of money.

8.—Materials of Construction.

Brickwork and Concrete.—Most of the usual materials of construction have been adopted for tunnel linings, and may be used if they suit the existing conditions. A tunnel driven through non-water-bearing rock may not need any lining at all, or perhaps only a facing of concrete to improve the appearance and to protect the rock against weathering. A tunnel constructed in dry soft soil may be lined with brick or concrete, providing that no shield-ram pressures have to be supported on the lining and that no soil pressures are likely to occur until the mortar or concrete has set. This indicates the use of brickwork or concrete,

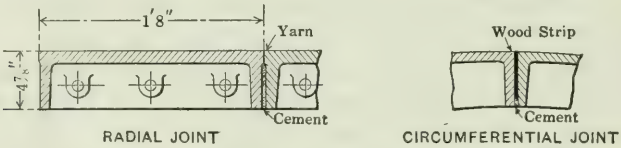
particularly as a secondary lining. The primary lining may be either a temporary timber construction or a permanent lining of timber, concrete blocks or metal. In the first case the masonry lining should be able to carry the full external loads; in the second case the masonry lining is used either to carry part of the stresses or to provide a smooth interior for the tunnel or to protect the primary lining. The use of precast concrete blocks placed without mortar will be discussed later.

Wood.—On account of its lack of durability, the use of wood is generally limited to temporary lining, but for this purpose it is quite common, from the timbering of a heading to the lining of full-size tunnels. For water-carrying tunnels in water-bearing soils, however, wood may be a suitable material. A special case where wood is indicated as a proper material for tunnel lining, is that of discharge tunnels of power houses, as it has been the experience that concrete or mortar disintegrates under the action of the discharge water. A wooden tunnel lining may be either arch shaped and made of short lengths of timber with circumferential butt joints, or it may be made rectangular, subjecting the timber to bending stresses. On account of the softness of this material, an internal wearing surface should be provided, either of masonry or of renewable planks.

Cast Iron.—Under proper conditions, cast iron is a material eminently suitable for use as tunnel lining on account of its rugged, durable character and its ability to carry compressive stresses.

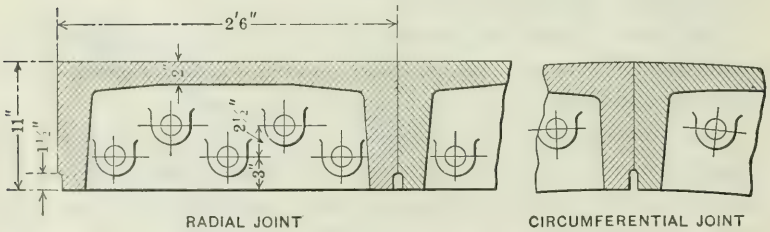
The tunnel shield, and incidentally the cast-iron lining of the type generally used, was originally invented by Marc Isambard Brunel and patented by him in 1818, but the segmental iron lining was not adopted until 1869 when it was used for the Tower footway tunnel, with an outside diameter of 7 ft. 1¾ in., at London under the Thames River. The lining is made up of segments, bolted together, forming a ring, and the adjacent rings are also bolted together. Fig. 9 shows sections of the cast-iron lining used for the Central London Railway in London clay, which is a stiff non-water-bearing clay. The outside diameter of this tunnel is 12 ft. 6 in. and no secondary lining is used. Fig. 10 shows sections of the cast-iron lining used for the greater part of the Pennsylvania Railroad Tunnels in the silt under the North River at New York City. The outside diameter of this lining is 23 ft. and a secondary lining of concrete was used, making the total thickness of the tunnel lining 2 ft.

It will be noticed that the method of making the joints is different in the two tunnels shown. In the Central London Tunnel the surfaces of the flanges were not machined. The radial joints were made by packing a creosoted wood strip between the adjacent flanges and bolting them together. For the circumferential joints, on account of the necessity of sustaining the shield-ram pressure, the flanges were brought into contact, metal to metal, along the skin and a packing of yarn was placed at the bottom of the joint behind the bolts. The remainder of the joint was filled with mortar except where water was met, and here the usual mixture of sal-ammoniac and iron borings was packed into the joint. The radial joints were also finished with a pointing of cement or of rust caulking, as required.



DETAILS OF CAST IRON LINING FOR CENTRAL LONDON RAILWAY TUNNELS

FIG. 9.



DETAILS OF CAST IRON LINING FOR PENNSYLVANIA RAILROAD NORTH RIVER TUNNELS

FIG. 10.

In the Pennsylvania Railroad Tunnels all flange surfaces were machined so as to insure an exact bearing surface over the entire depth of the flange. A caulking groove about 1½ in. deep, however, was provided along the inside edge of the flanges. This groove was used to make the joint water-proof by filling it with rust caulking, and the bolts were water-proofed by packing grummets consisting of rings of yarn dipped in red lead under the heads and nuts of the bolts.

The main advantages of cast-iron lining are that it may be handled roughly without being damaged; that it may be procured in the shape

desired and in units of a size limited only by the facilities for casting and the weight that can be handled in the tunnel; that it may be erected and connected in a reasonable time, and that it may be made almost water-tight by caulking the joints and grummeting the bolts. The accuracy with which the faces of the flanges may be machined is nearly absolute and it is possible to erect iron thus machined to perfect shape. The efficacy of the lining, as a structure and as a water-proof envelope, then depends on its capability of resisting, without deformation, the stresses which come upon it.

Cast-iron lining of the type described forms an efficient structure in soils with a large angle of repose, if the diameter of the tunnel is not too large. The question of cost is not considered for the present.

The objections to cast-iron lining are as follows: The structure, in effect, is not capable of resisting stresses other than compressive. This is not so much due to the cast iron itself, which might be relied upon to carry a reasonable tensile stress, but to the method of fastening the segments together, which permits them to separate slightly under a bending stress. The result is that during construction in soil with a small angle of repose it is subject to various non-elastic deformations, and further the pressure of the compressed air causes, as has been stated before, bending stresses in the lining, producing distortion.

In soils of the character of the North River silt, deformations may occur due to temporary disturbance of the earth pressures on account of the pushing forward of the shield, particularly if the shield is driven ahead without taking in the full volume displaced. When such distortions occur, it is necessary to provide temporary bracing and support until the supplementary inside lining is in place, and this is expensive both in time and money.

In soil of moderately high angles of repose, immediately after the shield leaves the lining, this is surrounded with an annular space only partly filled with solid matter. This condition produces local pressures on the lining, causing it to deform; then again, when the air pressure is removed the conditions of equilibrium are changed and the lining is subject to further deformation.

The non-elastic deformation has another serious effect on the structure. The horizontal joints between the cast-iron segments, which are generally machined and expected to bear on the whole surface of

the flange, open up either on the inside or on the outside and thus not only is the strength of the ring as an arch greatly decreased but also the efficacy of the caulking is diminished. This is particularly important, because the caulking, in order to be effective, must be done under air pressure, and when the pressure is removed, deformation takes place, opening the joints and destroying the water-tightness. In a material such as North River silt, having tidal movements, an additional and periodical working of the joints may occur which will make the caulking ineffective, particularly in tunnels with no inside concrete lining. Leakage necessitates continuous expenditure for pumping, and in a material like North River silt, even a minute leakage produces a settlement of the tunnel, affecting its gradient.

The skin of the cast-iron lining has to support the thrust of the shield rams. On account of the construction space needed for the rams, their pitch-circle must be several inches inside of the skin-circle. This causes bending stresses in the rams and in the lining and, in large tunnels, results in bending and jamming of the rams, and breakage of the lining.

These considerations suggest the following changes in the design of cast-iron lining. The depth of the circumferential flanges should be made large enough to insure compressive stresses only in the lining or at least tensional stresses no greater than can be carried safely by the cast iron and the connection between the cast-iron segments.

The sections forming a ring should be connected so that non-elastic deformations cannot occur, and longitudinal ribs should be provided to support the thrust of the shield rams.

By constructing stress polygons for tunnels of various diameters it has been found that, if the pressure at the top of the tunnel is 4 000 lb. per sq. ft., the weight of the soil 120 lb. per cu. ft., and the angle of repose is 2° 52′, the maximum eccentricity, e, of the circumferential force in the lining, when the outside diameter lies between 12 ft. and 36 ft., may be expressed empirically by

$$e = \frac{d^2 \sqrt{d}}{480} \qquad (10)$$

where d is the outside diameter in feet and e is expressed in inches.

It is desired to make clear that the soil characteristics and weights of lining from which this formula is derived have been deliberately

chosen to accentuate the effect of a heavy soil together with a low angle of repose. The flanges and thicknesses of lining given in the following pages, therefore, are exaggerated. In soil of the character of North River silt, for example, the eccentricity will be found to be considerably less.

If the depth of the flange is made equal to $4e$, the tensile stress will not be greater than may be brought safely upon the cast iron, and the connections between the segments of the lining may be made strong enough to carry this stress. The depth of the flange will accordingly be as follows:

Outside diameter, in feet	12	18	24	30	36
Depth of flange, in inches	4	12	24	41	65

The inside diameter, Δ, is determined by

$$\Delta = d - 8e$$

By differentiation, the maximum value of Δ may be determined as 26.2 ft. In other words, in order to obtain a cast-iron lining not subject to non-elastic deformation under the conditions assumed, the inside diameter cannot be greater than 26.2 ft. Before this limit is reached, however, the size, weight, and number of the segments of the lining will have become unwieldy.

It is quite possible, however, to construct tunnels with a cast-iron lining having a flange depth less than that required to make the lining self-sustaining; in fact, the primary lining may be and has been made of a thin metal plate only. When the lining is not self-sustaining, it must be temporarily supported during construction and a secondary lining must be provided. The question of which method to adopt is essentially one of economy, but it must also be kept in mind that when the primary lining is not self-sustaining, the water-proofing becomes more uncertain and expensive.

It has been found, by plotting the weight of cast-iron linings of tunnels actually constructed in soils of this character, that the weight may be expressed by the equation, $W = 22d^2$, where W is the weight, in pounds per linear foot of tunnel, and d is the external diameter, in feet.

Precast Concrete Blocks.—As previously stated, concrete is not, except under special conditions, a suitable lining material for a shield-

driven tunnel, except in the form of precast blocks. In this shape it is well suited for tunnels of any diameter when the soil is dry and has a large angle of repose, and it may be used in tunnels of small diameters in water-bearing soil with a small angle of repose, providing it can be made water-tight; but, according to the foregoing stress determination, in the present state of the art it is not suitable under any other conditions.

The advantages of the precast concrete blocks are that they may be made of any shape and size desired, at a reasonable cost, and that they may be handled with ordinary care without being subject to serious damage.

The disadvantages of precast concrete blocks are, as a whole, the same as those of cast-iron lining, but to a greater degree; namely, they cannot resist other than compressive stresses and the water-proofing is uncertain in its efficacy.

It has been stated previously that bending stresses occur during the construction. These stresses the concrete-block lining cannot resist, even though interlocking arrangements are provided between adjacent segments, because it would be a practical impossibility to cast the blocks so as to secure the absolute fit necessary to prevent any motion in the joint. The result is, therefore, that the tunnel will distort and the joints open, when bending occurs, thereby throwing the full load on a small bearing surface either at the inside or outside edge, and producing a grave possibility of spalling off the edges of the blocks.

Owing to their bulk, the concrete blocks would have to be rather short, particularly in tunnels of moderate or large size. Consequently, the taper of the individual blocks would be slight, and it is entirely within the possibilities when the tunnel distorts and the joints open, that the block might be free to slide in under the external pressure, particularly when the outside edges are spalled off.

In order to make a tunnel lined with such blocks safe during the construction period, it would be necessary to make arrangements on the following general lines: To provide the blocks with steel rings securely fastened into them and during the time the air pressure was on to fasten tie-rods to these rings in the directions where increases of diameter might occur, and at the same time to support the lining with struts in the directions where the diameters might decrease. Such temporary support however would be expensive and interfere with

the work of construction. The stresses in the structure under normal air pressure might be taken care of by making the thickness so great that tensile stresses would not occur. As will be shown hereafter, this is only possible up to a certain inside diameter. The tensile stresses in the permanent lining might also be relieved by introducing proper permanent internal bracing of the lining, which, however, again would add to the cost of the structure.

The concrete blocks might be made reasonably water-tight by the addition of some water-proofing compound. The joints between the blocks could be water-proofed by means of water-proofing material, but in order to make such water-proofing efficient, it would have to be carried out with the greatest care at each individual joint before the next block was placed, thus retarding progress at the period of construction, when speed is of paramount importance for economy.

The efficacy of the water-proofing, however, would depend on the absolute prevention of distortion of the tunnel.

As stated above, Equation 10 expresses the eccentricity of the circumferential force. In order to avoid tensile stresses, the thickness of the lining must be not less than $6e$. This would require the following thickness of the lining:

Outside diameter, in feet.....	12	18	24	30	36
Thickness of lining, in inches.....	6	18	36	61	97

It would appear, therefore, that the thickness of this lining becomes so great as to be impracticable when the outside diameter of the tunnel is more than 24 ft. Furthermore, the inside diameter should not be increased beyond 20 ft., because the inside diameter, Δ, is $\Delta = d - 12e$, and by differentiation, the maximum value of Δ is determined as 20 ft. In other words, in order to obtain a concrete-block lining not subject to non-elastic deformation under the conditions assumed, the inside diameter cannot be greater than 20 ft. unless inside bracing is provided.

Structural Steel.—So far, all the usual materials of construction have been considered except structural steel, which up to the present time has been used for tunnel lining only to a very limited extent, in spite of its many apparent advantages. The most probable reason for this is that the standard material for primary lining for subaqueous tunnels, cast iron, has proved itself generally satisfactory, although in

soils having a low angle of repose it has become apparent that the limit of possible size has nearly been reached.

Steel has been used as primary lining for tunnels under the Spree and the Elbe Rivers in Germany. In both cases the steel lining was used as an envelope only, inside of which the permanent and supporting lining was constructed. Light wrought-iron plates were used for the old North River Tunnel, commenced in 1874. Steel has also been used

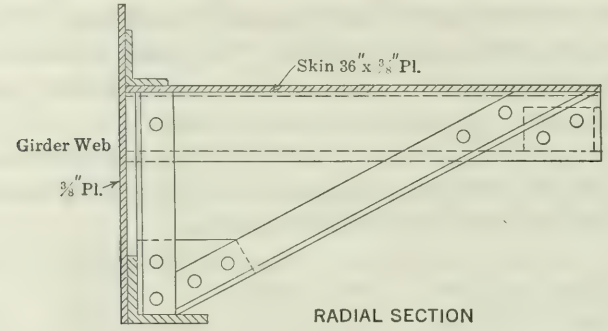


FIG. 11.

for a short length of the intake tunnel of the Hudson and Manhattan Railroad power-house at Jersey City, N. J. In this case, the steel lining was made of 13-in. ship channels, bent to the curvature of the tunnel and provided at the ends of each section with angle flanges, producing sections of steel similar to the usual cast-iron sections. These sections were erected and bolted together, following the procedure of cast-iron lining. This tunnel was provided with an internal wearing surface of masonry.

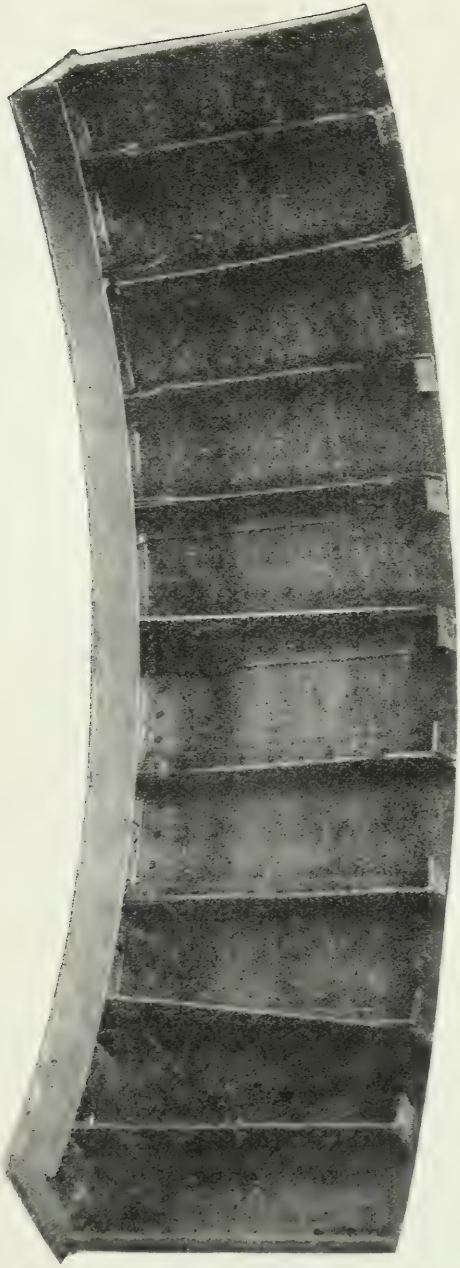


FIG. 12.—MODEL OF A STRUCTURAL STEEL SECTION FOR 18-FT. TUNNEL.

None of these types of steel lining is self-supporting but requires a secondary carrying lining. For the purpose of investigating the possibilities of a self-sustaining steel lining, the writers have, therefore, attempted to design such a lining. Fig. 11 shows its general character. It consists essentially of circular plate girders, spaced at intervals, supporting the steel plate envelope or skin. The girders are intended to be able to carry the forces that may come upon them without non-elastic deformation, but, as they necessarily must be brought into the tunnel in sections, they must be spliced. The compressive stresses will readily be transmitted through the machined abutting faces, and the tensile stresses may be transmitted through splice plates connected to the adjacent sections by means of tight-fitting pins or tapered dowels. Fig. 12 is a photograph of a model of a section for a tunnel 18 ft. in diameter.

The advantages of such steel lining are, first, that it may readily be made strong enough to carry without non-elastic deformation the forces to which it may be subjected, and, second, that it may be made absolutely water-tight by means of electric welding.

The principal criticism that might be made against the steel lining is probably that of possibility of corrosion. All the evidence, however, that so far has come to the knowledge of the writers appears to indicate that steel embedded in soil under conditions similar to those to which a tunnel lining is subjected does not corrode. More light on this matter would be desirable. Some evidence is afforded by the shield of the North Tunnel of the up-town tunnels of the Hudson and Manhattan Railroad. The skin of this shield consisted of a cylinder made up of two thicknesses of $\frac{5}{8}$-in. steel plate. The diameter of the cylinder was 19 ft. 11 in. and its length from cutting edge to end of tail was 10 ft. 6 in. This shield was erected in 1890 and was driven forward about 2 000 ft. in a little less than a year. It is not known whether the shield was painted or otherwise protectively coated before it started on its journey, but it is reasonably certain that any coating it may have had was rubbed off after traveling 2 000 ft. through the silt.

The work on the tunnel was abandoned in 1891, and the shield was left encased in the North River silt about half way across the river. Inside, the shield was filled with water and muck. In 1902, eleven years after the tunnel had been abandoned, it was cleaned out. The shield was found to be in good condition and, after repairs to the

machinery, was used to drive the tunnel the remaining distance across the river where it arrived and was dismantled in 1904, except the outer shell, which was left in place, the cast-iron lining being erected inside it. In other words, a steel cylinder with an unprotected surface of about 750 sq. ft. remained embedded in North River silt for eleven years without being impaired for use.

By making designs for a lining of this type for several different sizes of tunnels under the same assumptions as to soil characteristics

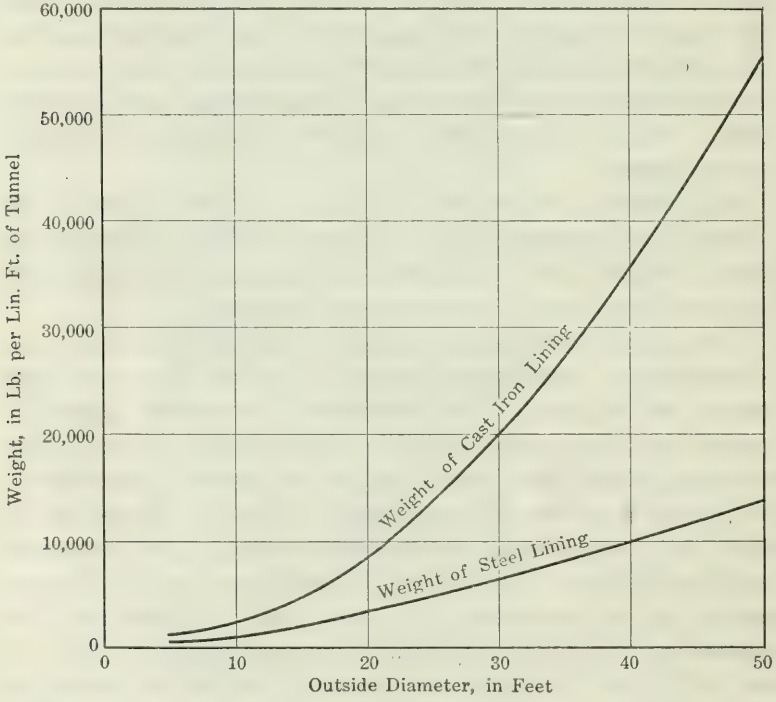


FIG. 13.

used previously, and by selecting the girder depth, expressed in inches, equal to the outside diameter, d, in feet, divided by 1.2, it has been found that the weight, W, of the lining, in pounds per linear foot of tunnel, may be expressed by

$$W = 4\,d^2 + 90d \qquad (11)$$

It will be found that weights of steel lining based on this formula, when compared with weights of cast-iron lining, range from 59% in the case of a 10-ft. tunnel to 26% for a 50-ft. tunnel.

Fig. 13 shows these weights plotted for tunnels up to 50 ft. in diameter.

9.—Secondary Lining.

The purpose of the secondary lining of a tunnel may be either to act as a carrying member, to produce a smooth inner surface, to give additional weight, or to protect the primary lining from corrosion. It generally consists of either brickwork or concrete. It has been stated before that in order to avoid troublesome deformations in a tunnel driven under air pressure, the primary lining should be designed so as to be able to carry the stresses to which it is subjected during the construction, as well as the permanent stresses, without non-elastic deformation. If that is done, no secondary lining is needed on account of strength and it might, if intended for that purpose only, be dispensed with.

As to the use of a secondary lining to produce a smooth inner surface, this is necessary when the tunnel is intended to carry water if the primary lining does not provide a smooth inner surface, for example, when the primary lining is constructed of metal. If the tunnel is intended to be used as a railroad or a highway, there does not appear to be any more reason to hide the metal construction than if the structure were a bridge.

10.—Cost of Tunnel Linings.

The previous sections of this paper have discussed the merits and demerits of various materials of tunnel linings from the point of view of serviceability without reference to the matter of cost. This feature, however, is of prime importance and often is a determining factor as to whether a tunnel is to be constructed or not, or if constructed, determines its capacity.

It was attempted, therefore, to formulate certain expressions by which the probable cost per unit of length of tunnels of any given size to be constructed through various kinds of ground, could be determined, if not absolutely, at least relatively.

The prices of labor and materials, however, usually are in a state of flux, and this is particularly true at the present time. Furthermore, the probable cost of labor and materials on any specific piece of work is so tied up with the local conditions which surround and attend that work that the conclusion was reached that such generalized and sweep-

ing formulas were more or less spurious and untrustworthy and might lead to erroneous results if applied without due caution and reserve.

It may suffice to repeat that, as far as the use of a tunnel is concerned, the internal cross-section is the measure of its capacity. As far as the stresses which will exist in the lining are concerned, the outside diameter is the ruling factor. Hence, in comparing one type of tunnel lining with another for tunnels of the same internal cross-sections under the same external conditions of ground pressure, assuming that the various linings compared have been designed so that the tunnel shall have equal strength in each case, the following factors should be brought into the comparison:

1.—The volume and cost of the excavation;
2.—The weight and cost of the lining material;
3.—The probable rate of advance;
4.—The permanence of the lining material;
5.—The possibility of making the lining water-proof.

11.—Conclusions.

From the previous discussion the following conclusions are reached:

1.—When the characteristics of the soil through which a tunnel is to be built are known, it is possible to determine the stresses in the tunnel lining.

2.—In order to obtain direct compressive stresses only in the lining of a circular tunnel, the diameter of the tunnel must be limited to a certain value which varies directly as the angle of repose of the soil in which the tunnel is embedded.

3.—In soil with an angle of repose producing bending stresses in a circular tunnel, it is possible to reduce, and sometimes to eliminate, the bending stresses by making the shape of the tunnel elliptical.

4.—If the total vertical downward forces are less than or equal to the total maximum passive upward forces the tunnel will not sink.

5.—If the sum of the total downward load and the frictional resistance is greater than the total active upward force the tunnel will not rise.

6.—The pressure of the compressed air in the tunnel during construction produces bending stresses in the lining.

7.—The shoving of a shield through plastic soils without taking in the full volume displaced by the shield induces temporary pressures in the ground which tend to deform and displace the lining.

8.—The lining should be designed so as to be able to carry not only the permanent stresses but also the construction stresses, which include bending, even though the permanent forces produce compressive stresses only.

9.—Consideration should be given, in each specific case presented, to the advantages and disadvantages, from an engineering viewpoint, of making the primary lining the entire carrying member, or of supporting the primary lining when and where necessary until the secondary lining has been placed.

10.—Up to the present, the rate of progress of a shield-driven subaqueous tunnel has been a function of the diameter of the tunnel. For each variety of ground this relationship is different, but in general the rate of advance increases rapidly from zero diameter up to a diameter of about 10 ft. and then decreases with increase of diameter.

11.—In soils with a low angle of repose, for any specific case presented, there is a limiting inside diameter (which may be determined for that case) beyond which the primary lining, whether of cast iron or of concrete blocks, cannot be designed to stand without internal bracing.

The subject covered is one on which there is still much difference of opinion and doubt. The writers hope that these notes may give rise to a discussion on the design and construction of tunnels which will help the engineer who has to deal with this most fascinating work.

DISCUSSION

Mr. Meem. J. C. MEEM,* M. AM. SOC. C. E.—This paper has been carefully prepared and contains much of value in the way of data and conclusions. In the speaker's judgment, however, it also contains, unfortunately, serious misconceptions as a basis for some of the conclusions given, especially, first, as to the characteristics of the Hudson River silt, and, second, as to the pressures exerted by it and other subaqueous soils.

First, as to the characteristics of Hudson River silt: Referring to the fact that a shield can be pushed into this silt without opening the doors, the authors state:

"That this particular soil is essentially a viscous fluid. Such a thing could not be done in clay or sand or a mixture of such. Observations on the jack pressures used to start an actual shield in movement through the silt confirms, when analyzed, the fact that the material has an angle of repose of less than 3 degrees."

Second, as to pressure, the authors state:

"This was done by analyzing many hundred observations of the external pressures on the tunnel lining, obtained by means of specially constructed and highly sensitive pressure gauges inserted in holes in the skin of the cast-iron lining."

Taking up first the alleged misconception as to characteristics, the error is probably due to two principal facts: That such samples as can be obtained and observed are apparently of an aqueous rather than a stable nature, and the further fact that soil into which a shield may be pushed without extraordinary pressure or without opening the doors points to soupy or at least a viscous fluid.

Although it is true that this cannot be done in sand or gravel, it can be done in clay or in a mixture containing a large percentage of clay and a small percentage of fine sand. In fact, this is the composition of the soil under the Hudson River below the first layer of fine organic deposit. In filling a void under a pier foundation under the United States Post Office recently, a specially constructed jack was used by means of which clay amounting to about 1 500 lb. was jacked into the void under a pressure of about 1 200 lb. per sq. in. It was found that sand up to 40% could be mixed with the clay without interfering with its susceptibility of flowing under heavy pressure. When it is realized that a shield can be jacked into the stiffest clay or even into solid lead, under proper conditions, it should be understood that soupiness or extreme viscosity is not essential to this condition, but its visualization is only a sop thrown by and to our finite minds in order better to comprehend its action.

* Brooklyn, N. Y.

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Assuming, for instance, that a shield with a closed frontal area of 350 sq. ft. advances into this soil under a pressure of 1 400 tons, and that 700 tons are required to overcome the frictional resistance, the force displacing the soil is 2 tons per sq. ft. While with all allowances it is probably much greater than this, it may be assumed that 2 tons is the minimum. If, now, a puddle of clay is made up in a pit or large vat and a pressure of 2 tons per sq. ft. is brought to bear upon it at the base of a hollow pipe set in the mixture, it will be found that nothing short of a pretty stiff mixture will resist the pressure so applied.

Another fallacy is that compressed air changes the texture, or at least the density, of the soil to which it is applied, thereby hardening it or stiffening it. As a matter of fact, it simply drives out the water and that the water can be driven out shows that in its normal state the pressure due to it is independent of that of the soil.

It should be realized that air does not support the soil, but prevents disintegration by driving out the water or holding its inflow back. In its normal bed, with no percolation or flow of water through it to disintegrate and soften it, this soil probably would appear exactly as if under air pressure, the only difference being that the air replaces the water.

The fact that the old Hudson Tunnel was driven for a long distance under the Hudson River without a shield—with an open face—and that the material under air stood up well and was very stiff, and that the bottom did not tend to rise, should convince the most skeptical that this silt has not normally the soupy or aqueous properties with which it is generally credited. There is also no reason to doubt that if this soil was as soupy as it is sometimes conceived to be, the shield could not be prevented at times from running ahead under its own air pressure.

As to the angle of repose which is given by the authors as being less than 3°, it is impossible for the speaker to conceive of a soil having an angle of repose anywhere near as small as 3° without being aqueous, and, therefore, having no angle of repose. The error of crediting aqueous materials (which can have no angle of repose, just as water has none) with flat angles of repose is frequently made. In all probability this silt in its natural bed, when not subject to the influx or outflow of water, will stand at an angle nearer 60° than 3°; it is even probable that if the bed of the river is dredged below the top layer of organic deposit, the silt will be found to stand at an angle of at least 45 degrees. This, however, will not necessarily determine its angle of repose in any case, for, although the water pressure tends to prevent the disintegrating outflow of water through the soil, it also keeps its surface in a softer and more pliable condition than if unexposed in its natural bed. The best test to deter-

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mine its normal angle of repose is to note its action under compressed air. In tunneling in subaqueous sand, for instance, it will be found that under compressed air its action is similar to that of a normally dry sand under normal air pressure and that it will have to be sheeted and braced in the same way; further, its angle of repose is the same under both conditions.

In tunneling in clay or silt composed of clay and fine sand (and there are numberless instances of this having been done without the use of a shield, especially in the Lake cities), the same relative conditions will prevail, except that the cohesive qualities of the soil act in conjunction with its arching properties, rendering sheeting and bracing less necessary than in sand. That is, in the speaker's judgment, the same sheeting and bracing would be necessary in subaqueous soil under air as would be required for the same kind of soil under normally dry conditions. Observations of the tendency of the soil to break down under vertical cutting should determine in both cases the normal angle of repose. As heretofore stated, the speaker's judgment is (which in this case, perhaps, may be only an educated guess) that this angle will probably be about 60 degrees. This will be referred to later.

As to the distortion of the tunnel, it is somewhat difficult to believe that an elongation of the vertical axis actually takes place, but if (or where) this is authenticated and not noted as the result of unchecked observations, it may be accounted for as follows: The tube is surrounded by a more or less aqueous or soupy envelope between it and the more stable material. This being heavy, it tends to flow to the bottom and thus keeps the tube from subsidence, while the greater resilience of the material displaced sideways reacts more quickly than the vertical displacement (which has more room for expansion) and thus elongates the tube in this direction.

As a matter of fact, however, the records show that the Pennsylvania Tunnel settled during construction nearly 4 in. and that there was a tendency for the sides to press out into the voids left at each side by the passage of the shield. That is, its history seems to be one of vertical props and lateral tension bars—also, in spite of these, of occasional cracked plates—and this is the history of virtually all tunnels, except those backed up by some solid filler at the sides which may be supplemented by grout. In the Battery Tunnel, on Joralemon Street, Brooklyn, N. Y., where air was not used, the grout immediately flowed to the bottom and, although it prevented the tube from settling, it left voids at the sides into which the plates at once spread. This resulted in the cracking of many of the crown plates, though it is interesting to note that these cracked plates carried (when the sides had found solid backing) the load which had apparently cracked the good plates.

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The apparent remedy is, in all cases, to apply some solid filler, such as clay or gravel, directly after the passage of the shield and follow it with grout. This will tend to prevent distortion, eccentric loading, and other ills now attendant on construction.

Much importance appears to attach itself to the fact of the rise and fall of the Pennsylvania tubes, due to the fluctuation of the tide. This is probably due to the resilient action of the more or less compressed material and not to any condition of fluidity. While, from entirely different causes, the fluctuation may be compared with the action of a tube embedded near the surface in a mass of solid iron which would fluctuate with the temperature changes of the iron, as well as with its expansion and contraction, if due to pressure and resilience exerting the same stresses as temperature changes.

What the speaker considers to be errors in pressure are due largely to the interpretations of some of the characteristics of the soil, as just noted, but principally to the interpretation of the pressure-gauge readings. He has often pointed out the fallacy of attempting to interpret the pressure on the whole area of a structure by pressure readings on gauges through orifices in the lining. To attempt to do this in normally dry soil is comparable to attempting to estimate whether the chamber of an hour-glass is a cubic foot, a yard, or an acre by placing one's fingers or a gauge over the orifice, when it is obvious that orifices of the same area will show the same pressure for all sizes of chambers. Gaugings through linings of structures in subaqueous soil are almost equally misleading, for while the gauge may show full pressure due to the hydrostatic head of water in each of a dozen instances, the tunnel may be resting on solid soil bottom from which no pressure results on the bottom side and a separate and distinct pressure (to be noted later) from the soil at the top.

If, however, the gauges indicated full hydrostatic pressure and it was distributed over the whole area, then the stress computation would be simplified, though the actual problem would be greatly increased, being principally the prevention of flotation. This latter, of course, would occur when the upward pressure exceeded the downward pressure plus the weight of the structure. It would not be dependent in any way on the flatness of the angle of repose. Again, if, as is sometimes erroneously suggested, the soil exerts a fluid pressure equivalent to a fluid of a specific gravity of 1.6 (or an aqueous pressure of 100 lb. per sq. ft.), then the air pressure required, for instance, at a depth of 90 ft. would be about 60 lb. instead of 38. As a matter of fact, the air pressure required has usually been below the normal hydrostatic head in Hudson River tunnel operations.

In the case of these tunnels, had the pressure on the bottom been as was shown by the gauges, this bottom would undoubtedly have come up under the air pressure used—particularly in the old tunnel.

Mr. Meem. Where a bottom gauge shows a pressure greater than the normal hydrostatic head, it is probably due to settlement of the tube pressing the gauge into the soil, and when the gauge at the top or sides shows a greater than normal pressure it is due to the blanket-like action of the soil, which may press on an isolated gauge with greater pressure than the unit pressure of the area impinged upon.

The speaker understands that in many cases the soil into which these gauges were set had first to be stirred into activity before the

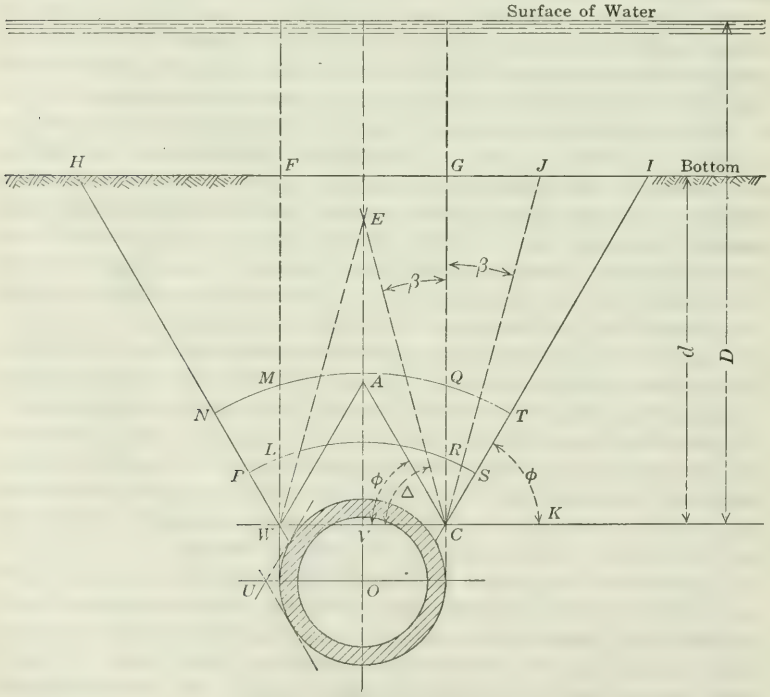

FIG. 14.

gauge would become operative; he has also heard of instances in other operations, where gauges were juggled to make them "read correctly."

In fine, the speaker believes these gauges will not show correctly anything except the hydrostatic head, which is always known, while the actual area of the pressure, as well as the pressures of the soil, will be left absolutely undetermined. He believes, further, that better results can be had from small models reproducing relative conditions and giving pressure readings over full areas than from interpretations of all the gauge pressures ever made.

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Under these conditions, normal and eccentric pressures, hydrostatic pressures, buoyancy, etc., can be determined definitely and correlated to the conditions of service.

The speaker has always held that pressures on walls, trenches, tunnels, etc., can be determined from the practical and rational interpretation of well-known or observed physical laws. His own method of determining the pressure on tunnels is as follows:

First, it should be obvious that soil and water cannot exert pressure on the same area at the same time. If, for instance, a column of impervious clay stands on the bottom of a tank full of water, there is no water pressure on the area occupied by the bottom of the column. The weight on the tank bottom then is the area of the water pressure less the least cross-sectional area of the column plus its weight, which is equivalent to saying the weight of water plus the weight of the column less the weight of water displaced.

If the column of clay is suspended so that it is in contact with, but does not stress, the tank bottom, the weight on the tank bottom will then be the weight of water over the whole area less the area of the column.

If mud or soil is placed in a tank filled with water and the tank is deep enough, the soil will eventually reach sufficient depth to arch and relieve the bottom of a part of its load. This can be demonstrated by any one on a small or large scale.

Referring to Fig. 14:

Let ϕ = the angle of repose of the soil in its normal bed;

$$B = \frac{90 - \phi}{2} = 45° - \frac{\phi}{2};$$

$$A = \phi + B = 45° + \frac{\phi}{2}.$$

If, for a moment, it is assumed that $FWCG$ is a sheet-pile cofferdam and that IC is the plane above which all soil would slide away if left unwatered and unprotected (that is, the unwatering should precede the operation of determining the angle of repose by allowing it to slide), then it must be assumed that the weight of some of the soil in the area GCI is transmitted as thrust against GC and some of it rests on the plane IC.

The speaker arbitrarily assumes that the plane JC bisects this area and that all the area GCJ exerts its pressure against the wall GC.

Reversing the condition, as shown, all the weight of ACV must be carried by the tunnel, by the same assumption as before, and all the weight of ACE also is carried by the tunnel. Above that, all the soil has become self-sustaining by reason of the development of

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its arching properties. This may be shown more clearly by the lines *PLRS* and *NMQT*, within which are developed the self-sustaining arch stresses. The curve *NMQT* is drawn so that the area *WMQC* will equal the area *WEC*.

While it is believed that the depth *EV* is greater than necessary to develop the sustaining arch, the speaker has always used this figure, as noted.

That is, although the soil will arch itself along the lines *NMQT*, it may probably require a height of soil above *WC* equal to *VE* in order to relieve the tunnel of the stress of the soil outside *WEC*, or its equivalent.

This height *VE* is found to be $\frac{L}{2}$ tan. $\left(45° + \frac{\phi}{2}\right)$, or $\frac{L}{2}$ tan. Δ, where L is the outer diameter of the tunnel.

Assuming, then, that the pressure of the soil is thus accounted for independently of the water pressure, it is evident from the foregoing that it is necessary to consider water pressure due to the full hydrostatic head, but only for that percentage of the area not pressed on by the soil.

It is probably not far from a correct assumption to state that this area is 30 and 40% of the whole, but as a safe factor the speaker has always taken 50% of the total as the practical area of the water pressure.

The pressures at the sides and bottom must also be considered. The side pressure (except during construction, when there may be an increased pressure due to the surge following the passage of the shield) is the thrust of the area between the tunnel and intersecting planes parallel to the planes of repose and tangent to the outside of the tube at the upper and lower quarters. It may be computed approximately as the weight of the soil in this area divided by the tangent of the angle of repose, that is,

$$P_s = \frac{A_s W_s}{\text{tan. } \phi}.$$

In view of the fact, however, that this pressure is much less than the vertical pressure of the soil, and that in its final position after construction the soil tends to resist displacement of the structure rather than to exert unbalanced pressure, the structure should be designed to resist at any point the maximum static pressure of the soil and water combined. Such allowance as in the judgment of the engineer may be necessary can be made for the exigencies of construction, but the effort should be directed toward reducing those exigencies rather than by adding a large percentage to the size, and thereby to the cost of the structure.

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The maximum pressures are thus found to be:

$$P = \frac{L}{2} \tan. \measuredangle \times \frac{L}{2} W + \frac{62.5}{2} D L$$

$$= \frac{L^2}{4} W \tan. \measuredangle + 31.25 D L$$

in which

P = total pressure on upper half of tunnel;
L = outer diameter or greatest width of tube;
W = weight of soil;
D = depth of water;
d = depth of soil to tunnel.

This is true for those cases where VE is less than d.

Where VE is greater than d:

$$P_2 = L d W + \frac{62.5}{2} L D$$

$$= L (d W + 31.25 D).$$

Where VE is so shallow that the possibility of full hydrostatic pressure is present, more especially during construction, then P may equal $L \times 62.5 (D + L) = P_3$.

The engineer is always justified in designing to meet the maximum pressure as between P_2 and P_3, but he is not justified in designing against a pressure represented by the total weight of soil plus the total weight of water, nor is he justified in assuming an aqueous pressure due to liquid soil of greater specific gravity than water, when the fluid medium is water.

An interesting photograph (Fig. 15) shows the difference in soil in its normal bed and exposed. This soil underlies Trinity Place in New York City back of Trinity Churchyard (which acts as a reservoir for it) and is the nearest approach to quicksand the speaker has ever seen. When exposed it is almost fluid and flows freely. A man will sink into it to considerable depth and it can only be handled by compressed air or elaborate methods of drainage. When drained it will stand at an angle steeper than 45°, and piles driven into it (when undrained and in its normal bed) only a few feet below the base of operation will bear, after some initial settlement, a sustained load of from 10 to 25 tons. The photograph shows 12-in. steel sheet-piling which was driven into this soil and met with such resistance due to stiffness and tenacity that it was distorted as indicated. The actual material into which it was driven is the same semi-fluid soil as that shown in the photograph. A 14-in. pile bearing some 6 ft. below the level shown is sustaining its portion of the surface and sub-surface structures with the ordinary street traffic, including trolley cars.

Mr. Meem. As to the question of rust of steel or iron disintegrating the exterior lining of subaqueous tunnels, the speaker believes there will be no deterioration whatsoever, or at best a small film of "protective rust." Since rust causes approximately tenfold expansion and any expansion will cause pressure, the initial rusting, if it takes place, will be pressed upon the metal and act as a protective coating. In this connection he presents Fig. 16, a photograph of light tunnel plates taken from the 15-ft. Greene Avenue sewer built on Hanson Place, Brooklyn, N. Y., in 1893, by the Anderson and Barr pilot method. They were removed in 1917 by the Degnon Contracting Company, and after 24 years in normally dry soil show almost no deterioration from rust. It is interesting to note that where voids were left by the moving of the plates at the rivet heads, these voids have filled with rust. This tends to show what the speaker believes to be true, that rust, like grout, will not expand into the normal voids of the soil, but will fill those artificially made.

In conclusion, the speaker believes that the normal pressure on the tunnel in place is definite and determinable and that the exigencies of construction should be eliminated as far as possible by following the shield with an envelope of clay or gravel, supplemented, if necessary, by grout, and that if this is done an allowance of not more than 25% should be made for contingencies.

The speaker does not believe that large sums of money should be expended to guard against remote contingencies, such as, for instance, the possibility that a tunnel if built even of solid cast iron could be displaced and probably disrupted by driving near it a larger and heavier tunnel.

It is essential that judgment based on observation and experience be allowed to become an important factor in all designing. When it is realized that the Battery Tunnel with, in some instances, only a 9-in. cast-iron ring with 2 in. of concrete in addition, and the McAdoo tubes with cast-iron lining only and no concrete, are permanent and stable structures and will always remain so—for even they meet with a large margin of safety the demands of adequate design for tunnels in place—engineers should endeavor in subaqueous designing to analyze the conditions that render them stable and not utterly disregard these plain facts.

For a concrete tunnel 30 ft. in diameter, 100 ft. at bottom below the water and 20 ft. at top below the bottom of the river, we would have 153 750 lb. per lin. ft. for silt weighing 100 lb. per cu. ft. Adding 25% for contingencies = 192 500 lb. At 600 lb. per sq. in. for concrete this calls for a 14-in. ring. The speaker, of course, would not even suggest the building of a 30-ft. tunnel with only 14 in. of concrete, but he does not doubt that the 14-in. ring in place would be absolutely stable. He believes that it would also not be economically

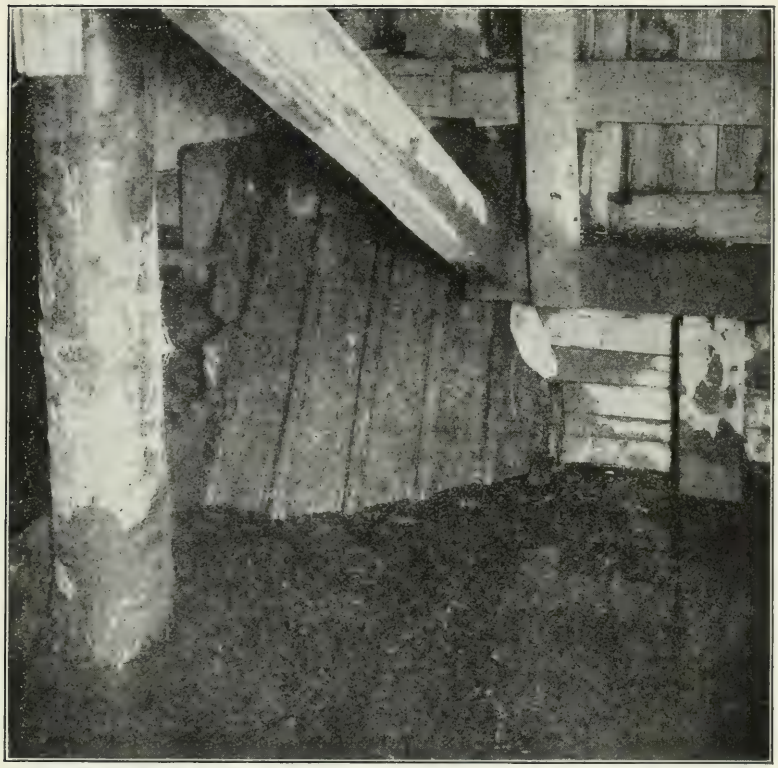


FIG. 15.—ACTION OF QUICKSAND ON 12-IN. STEEL SHEET-PILING.

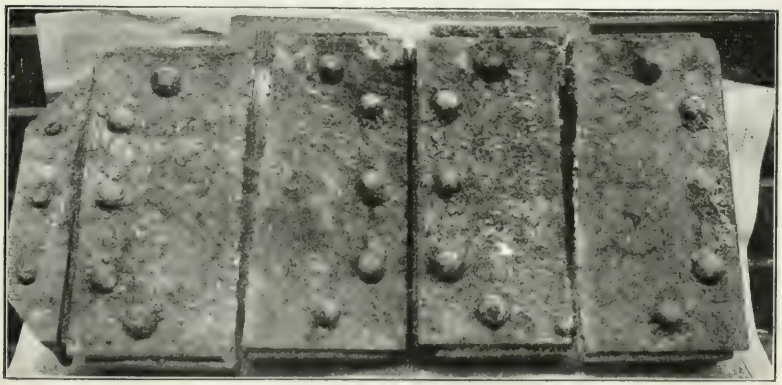


FIG. 16.—EFFECT OF RUST ON STEEL PLATES.

sound to increase this thickness to more than 30 in. under all conditions, including the safeguarding driving shield construction. Mr. Meem.

Finally, the speaker wishes to state his belief that fully 95% of the tunnel failures are due to improper backing of the sides. This also applies to back-filled structures.

E. G. Haines,* M. Am. Soc. C. E.—To avoid the necessity of any one else feeling called on to make the announcement, the speaker will state at the outset that although he has had experience in the construction of shield-driven tunnels in dry ground, and a limited experience in subaqueous tunneling, the latter has been confined to a position back of the lock, and he has not had any experience in compressed air. Nevertheless, he has had considerable experience with silt, and as this bears on some of the questions at issue in the paper, the following is submitted for consideration in this connection. Mr. Haines.

The authors state that the material encountered beneath the Hudson River is a silt with an angle of repose, so called, as low as 3° from the horizontal. The speaker assumes that the statement is intended to be taken literally, and that the authors imply that all the material above the bottom of a tunnel, lying also above the angle of 3° from either side, is, in its natural state, in an unstable condition, and exerts an influence and pressure on any body submerged therein. The speaker submits that were these conditions true, it would be absolutely impossible to build a tunnel within the material, to dredge and maintain a channel therein, or even to anchor a vessel in it.

When encountered in excavation, silt is usually called clay, because of its general appearance, and much trouble often ensues because of the error in classification. There is a great difference between the two materials, both as to origin and action. Clay is the result of decomposition of the feldspathic rocks, and such grit or sand as it contains is in the nature of an impurity and small in quantity. Such voids as it contains are not connected and it forms, as found in its natural bed, an almost perfect barrier to the passage of water. Water standing on it, or flowing over it, does not penetrate to any depth, but makes the surface slippery, like grease. It is plastic, however, and under sufficient pressure will flow without losing its cohesive quality, or developing fracture planes. Silt, on the other hand, is the result of mechanical disintegration of rock, in other words, it is rock flour, and contains only a limited quantity of clay to give it cohesion. It is practically all granular, and its density is due to its extreme fineness, the manner in which it was deposited, and the pressure to which it has been subjected. Silt has a relatively large percentage of voids, and these being connected, water passes through it as readily as sand. If, however, the water passes through under unbalanced pressure, or

* Brooklyn, N. Y.

Mr. Haines.

at too great a velocity, its low cementing qualities are quickly lost and it is converted into a mass closely resembling heavy paint, in which the water, as a vehicle, carries the grains in imperfect suspension. Silt will also flow under pressure—under certain conditions it will move by reason of its own dead weight in the same manner as other soils—but when this occurs the entire moving mass is broken up and all cohesion is lost.

The speaker has examined and compared samples of Hudson River silt with samples of other silts from different localities near New York City, both in dry ground and in other streams leading into the tidal waters in that locality, and he finds no material difference between the various samples, except such as might naturally be expected. Silt, being the finer pulverized material from the rocks in any river valley, differs in the mineral it contains in different localities. It also differs in fineness, and the silt of the Hudson River shows extreme fineness—it is finer than any other which the speaker has examined. Nevertheless, in their natural position, undisturbed, even though submerged beneath or filled with water, silts possess considerable cohesion and form a very compact mass.

Whenever one speaks of cohesion in soils, speaking first of dry ground, he is met with the exclamation "sand!", and speaking of wet ground, with the word "silt". Now, it is submitted that there is nothing mysterious or peculiar in the action of either sand or silt. In the condition found in Nature they both possess cohesion to quite a large degree. In fact, the cohesive element is far stronger than any other element tending to resist displacement—stronger than all others combined. Sand and silt stand at almost opposite ends of the entire range of materials classified as soils, one being the quite dry granular element, and the other the wet, compact plastic mass. The speaker presents photographs illustrating cohesion in each.

Fig. 17, reproduced from a photograph taken in 1916, shows a bank of sand cut vertically at the side of a subway excavation, where it crossed the line of a small drift excavated about 1905. This drift is beneath the surface of Fulton Street in Brooklyn, at Borough Hall, and was built for the diversion of a sewer, in connection with the old Brooklyn Subway in Joralemon Street. The work was done by J. C. Meem, M. Am. Soc. C. E., for Cranford and MacNamee, and was revealed by the excavation of the Clark Street line of the Dual Subway System. The photograph was presented to the speaker by Mr. Willis Thompson, Section Engineer for the Transit Construction Commission. It will be noted that above the poling boards of the drift there is a well defined arch in the sand. Mr. Thompson states that the material lying above the poling boards was finely pulverized, and that above the loosened material, as shown by the photograph, was a void, about 2 in. in width, between the solid and loosened ma-

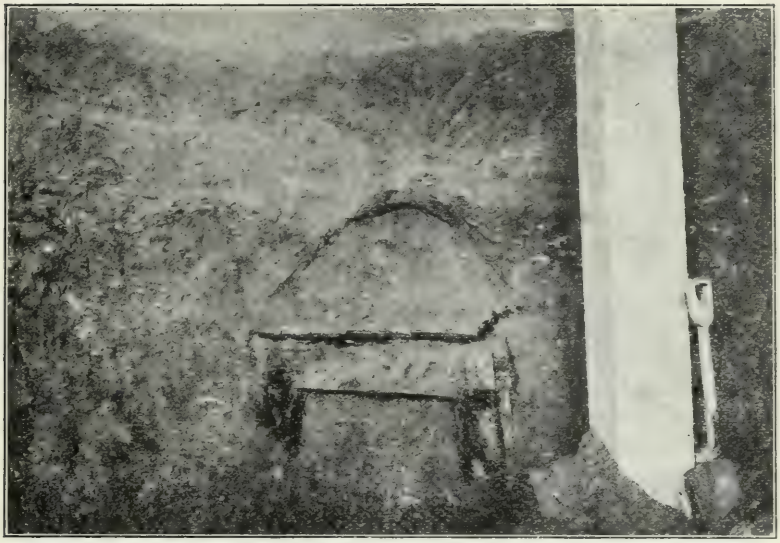


FIG. 17.—WELL-DEFINED ARCH ACTION IN SAND IN SUBWAY EXCAVATION.

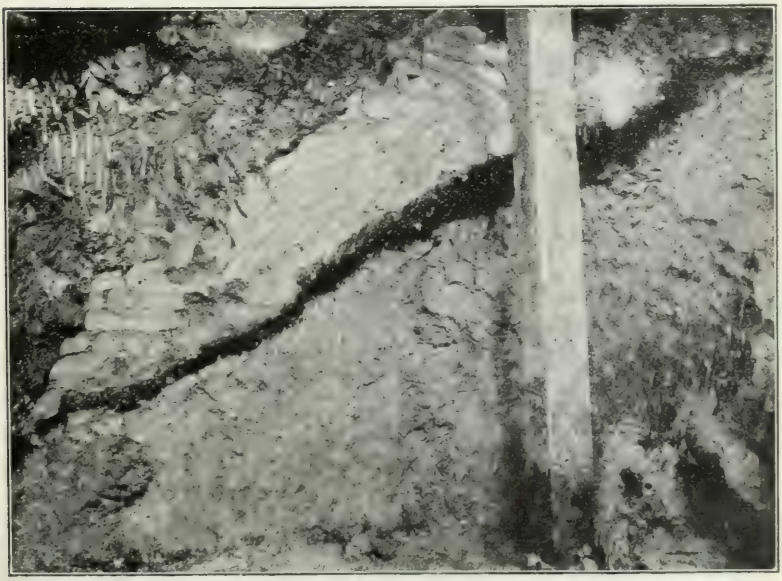


FIG. 18.—CHARACTER OF SILT IN SUBWAY EXCAVATION, NEW YORK CITY.

terial. The material through which the drift was driven was sand and gravel, rather loosely held together with cementing material, the firm overlying strata being fine sand. The top of the drift was about 8 ft. below the surface, and the point where the photograph was taken was approximately beneath the rail of the street surface railroad above. The width of the drift was 4 ft. and the rise of the arch exactly one-half the span. Notwithstanding the fact that this material had been carrying the street surface traffic of the busiest trolley line, probably in the world, for more than eleven years, the arch of sand had retained its integrity throughout. This photograph is a very good illustration of the cohesive strength and arching action always present even in relatively dry sand, as found in Nature. Mr. Haines.

Fig. 18 shows the side of a subway excavation in silt, a material called "clay" by the workmen; its character is clearly indicated. In the lower right-hand corner some large blocks may be seen and the marks of shovels and picks on the face of the bank are also apparent. The exposed face exceeded 10 ft. in height and was dressed carefully to line in advance of driving the sheathing planks. The heavy black line crossing the exposed face diagonally is organic matter and measured at its widest point about 12 in. The material both above and below the organic matter was silt. Samples were submitted for analysis, and it was found by using a micrometer under a microscope that the bulk of the material had a grain size of less than 0.01 mm., or even finer, to pure clay. Very few particles measured to 0.05 mm.

The extreme fineness of this material may be better gauged when it is stated that compared with it Portland cement is a very coarse material, and yet this silt, in turn, compared with Hudson River silt is quite coarse. Examination of the material shows that it is rock flour, with just sufficient clay to make it plastic. After dressing the bottom of the cut to grade, tests were made with a compression apparatus, by which the pressure on a single square inch of surface was determined, and it was found that, with a pressure of 50 lb., the instrument would hardly make an impression on the surface.

Notwithstanding the compact, solid nature of the material as exposed, after the completion of the excavation to grade, following a succession of rains, it was found that the workmen, in building the forms for concrete, churned it up into such a liquid state, that an average of over 1 ft. in depth had to be removed before it was possible to build the concrete bottom of the subway structure. Since the completion of the structure there has been no evidence of any settlement whatever.

Being carried in suspension and deposited by water, it is natural to suppose that silt would be in its softest state, and have the lowest angle of repose, at the time it was deposited, yet attention should be called to the fact that the angle at which this particular silt was

Mr. Haines.

deposited was 30°, as shown by the line of the organic matter. Furthermore, it must have remained at that angle for a long time, to allow of the accumulation of the organic matter, and, later, the deposit of silt above. It will also be noted that the angle of stratification repeats in the upper layer of silt.

The above example is of silt lying inland and extending so close to the surface as to be unaffected by ground-water, except that it is kept moist. The same material, however, was encountered some distance away, close to the East River, at a point where the street surface was about 6 ft. above mean high water. Ground-water level was about 1 ft. above mean high water. The excavation at this point was about 30 ft. in depth, about 20 ft. at the bottom being in the silt. The material was excavated in the usual manner by keeping the sheathing footed, and pumping from a sump to which several open ditches led. The bottom was at all times easily softened by working on it, but was readily dressed to grade. After the completion of excavation, work was suspended for an entire winter, and the cut was allowed to fill with water, which rose to a depth of more than 20 ft. The water stood in the cut for more than three months. It was then pumped down slowly. There was no evidence of great pressure, no bulging of the sheathing, and no rising of the bottom; in fact, nothing to indicate that the banks had been softened by the water, although all of it had passed in through the sides and bottom of the cut. There was a failure of each bank, as shown by cracks which developed in certain buildings, but the disturbance was limited to a distance equal to the depth of the cut, and even this was probably partly caused by the weight of the buildings.

The speaker has recently had opportunity to observe the action of water in the silt of the low lands adjoining the Passaic River, which not long ago flooded adjacent ground to a depth of several feet. At a distance of about 200 ft., observation in test pits showed that the ground-water level followed that in the river, receding, in some cases, at the rate of more than 1 ft. in 24 hours. Not far distant, however, are certain clay pits, with vertical banks, in which pools of water have stood to a depth of several feet for a number of years.

The authors state that the Hudson River silt is a material varying in weight with the depth below the river bottom, with an average weight of 108 lb. per cu. ft., and containing 55% of water by volume. The speaker believes that the latter statement is based on errors of observation, and that the error in the statement may be readily shown in this way: If every individual grain of the material were a perfect cube, and in contact with other grains only at the acute points of the corners, there would still be only 50% of water contained in the mass, and to increase that quantity it would be necessary that the faces of all the grains be made concave, or else that each individual grain be

carried in actual suspension. These conditions, it is known, are not true. As a matter of fact, the silt of the Hudson River is a most compact mass, due to the extreme fineness, the long interval of time during which it has been deposited, and the extreme weight which it carries on its upper surface. Mr. Haines.

The authors state that the material is readily compressed by an additional height of tide, and that the compression can be measured. From this, it is argued that the material is in a semi-fluid state. The speaker submits that the statements made directly contradict that supposition, for a fluid being practically incompressible, it is evident that the compression made by the tide is due to the elastic qualities of the silt considered as a solid. This is not peculiar to silt, but is true of all solid bodies, as under sufficient pressure, there is a reduction in volume, or, stated in another way, the volume remaining constant, the greater the pressure the greater the weight. This is confirmed by the authors' statements of increase of weight with depth.

The difficulty encountered in driving a shield through silt is due, not to the condition of the material itself, but to the conditions created by the driving of the shield. There is a vast difference between soil filled with water and water filled with soil. The second condition can readily be created from the first, by disturbance and erosion. The first condition cannot be created from the second, except after a very long interval of time with the water quiescent and after being subjected to a great deal of pressure. The process of driving a shield is such as to create a disturbance in the material. In dry ground, and the speaker believes also in wet ground, it is uncommon to use all the jacks for driving the shield. The result is that while the shield moves forward it is not, at all times, parallel to the line and grade of the tunnel, and has a tendency to disturb somewhat more material than that indicated by its outside perimeter. There is also a small space behind the shield which, in any case, requires to be filled. Due to the fact that water is always entering the tunnel near the bottom, or air passing out near the top, or both, there is constant change taking place by erosion, inward at the bottom and outward at the top. This has a tendency to bring to the bottom particles of the silt, which tends to restore the bearing at that point, but the erosion at the top is outward and with the pressures at the sides and bottom, there is naturally a tendency for the tube to rise. These conditions are, of course, much aggravated when the shield is driven without taking in the silt. In that case, there is an entire rupture of the overlying material for some distance on each side. As the rings are made tight, however, the action of the silt is downward at all points and equilibrium is restored. There is then a tendency to settle and spread laterally until resisted by the compression of the silt. These simple facts account for all the distortions and stresses indicated, as well as

Mr. Haines.

any theory, such as Rankine's conjugate stresses, which is based on conditions not actually found in practice.

The speaker, a number of years ago, had occasion to do considerable dredging in various tidal waters around New York City. The material removed was often so soft that it was difficult to hold it in the buckets, and even in the scows, and loads were lost a number of times by not being able to make the traps in the bottom close securely enough. Notwithstanding this, careful soundings taken after the dredging showed that the sides of the dredged area were very abrupt. In some cases the top portion outside of the dredged area could be observed at extreme low water. This material, which was mud and sand, was removed with a clam-shell dredge, and it was found that the depth of dredging was controlled by the compact silt beneath, for which that type of dredge was unsuited, as it would not penetrate to any extent into the silt.

There is nothing to indicate that the action of soil beneath water is much different from that of dry ground. The reason this condition might be expected the speaker will not repeat here, as it is already on record in his discussion of a paper on the "Bracing of Trenches and Tunnels",* presented by Mr. Meem. He would state, however, that in the past twelve years he has found nothing to contradict the suggestions he then made, but much tending to confirm them.

The authors state that when the Hudson River silt is subjected to air pressure, the silt has the nature of a stiff clay and its consistency may readily be varied by changing the air pressure. The speaker submits that this condition is due, not to the presence of the air, but to the absence of the water, and were it possible to exclude the water from the silt, or even to balance it exactly at all points, so as to restrain the water and prevent any erosion by reason of its passing through the silt, that most of the difficulty in subaqueous tunneling would be overcome. For that reason, while he has no desire to enter into any controversy as to the merits of different tunneling methods, he suggests that the proposition of using a shield in which it is possible to control, within narrow limits, the pressure at different heights, is decidedly interesting and a step in the right direction, and if the control could be extended a short distance, back to the point where the rings are caulked, the danger of soil disturbance from erosion would be largely eliminated. Whether the practical difficulties in the way of accomplishing this are too great to be overcome the speaker does not feel qualified to state, but if it is possible, then a shield-driven tunnel in the Hudson River silt might very readily be built much as one would conduct such operations in the same material in dry ground.

* *Transactions*, Am. Soc. C. E., Vol. LX (1908), p. 27.

PHILIP AYLETT,* Assoc. M. AM. SOC. C. E. (by letter).—Coming as it does on the eve of the construction of another great tunnel under the Hudson River, and in the midst of professional discussion involving similar problems related to that proposed great project, this paper is especially welcome and valuable at this time. It is evident that the authors speak from a knowledge born of experience, from success and failure met within their own experience and remedial effort; one has here a summation of it all, concisely and briefly recorded. Mr. Aylett.

In the entire field of engineering activity and enterprise none invites such delay, difficulties, anxiety, expense, and danger as the work of tunnel building by the pneumatic shield-driven process. The deeds and heroism of soldiers and sailors have been lauded in verse and song since time immemorial. We would not take one leaf from the laurel crown the world places on the brow of the man who risks his life for his country. He is worthy of all the admiration a grateful people can give; but peace as well as war has its heroes. The world is comparatively ignorant of the deeds and dauntless spirit of that other army of intrepid fighters who go down into the bowels of the earth and under the waters thereof to battle for the progress of civilization and uplift of humanity in the face of dangers lurking everywhere.

The writer's experience with shield-driven tunnels and pneumatic work has brought him face to face with many of the difficulties enumerated by the authors. In his effort to forestall or remedy such difficulties, considerable thought and experimentation has been directed toward modifications of existing practice.

The great stiffening effect and rigidity afforded to sheet metal by corrugations is common knowledge. Such corrugations afford like effects whether the metal used is flat, circular, or in other shapes. In fact, the strength and rigidity of sheet metal is increased to such an extent by corrugations or ribs that to-day it is used for purposes for which the plain flat sheets are entirely inadequate. This result is obtained with no increase of sectional area of the metal itself. The corrugated sheet metal culvert is an example. The most recent type of the garbage pail—which even defies the tender mercies of the collector—is another. Observation and experiment have led the writer to believe that corrugations, or, more properly, fins or ribs may be applied to the exterior surface of tunnel linings with decided advantages. These fins or ribs should be applied longitudinally and circumferentially. Although it appears that the provision of such exterior fins, in conjunction with the usual interior flanges, would afford sufficient strength and rigidity to meet ordinary conditions, still, in extraordinary cases, in order to insure longitudinal continuity of the usual interior flanges, some modifications of them would be necessary.

* Hollis, N. Y.

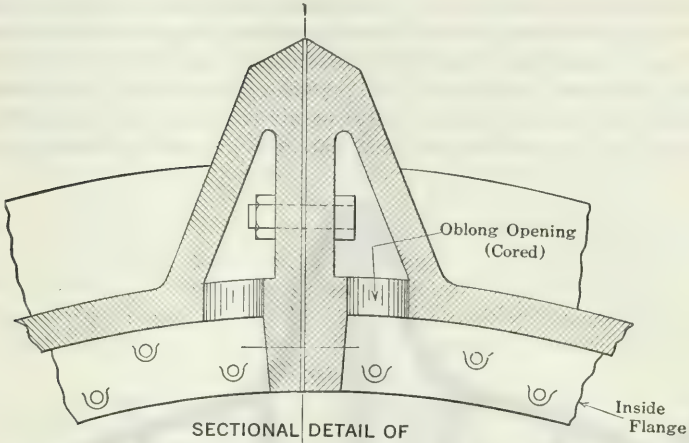
Mr. Aylett.

The addition of interior ribs suggested by the authors would be in this direction. The longitudinal ribs or fins proposed by the writer would protrude radially from the exterior surface of the lining and be located at regular intervals around its circumference in sets of four, eight, or more, as circumstances demand. (Fig. 19.) The width, area, section, and number of these exterior fins would depend on the tunnel diameter, character of soil penetrated, arrangement and power of rams, and air pressure. The same factors would influence the location and design of exterior circumferential flanges when such are required. It appears evident that such exterior protruding fins embedded in the surrounding material would exert a great stabilizing and stiffening effect on the lining, by virtue of their dual service as longitudinal stiffeners and soil anchors, while their location and design are such as to enable them to encounter and resist forces from any direction, whether from the inside or the outside.

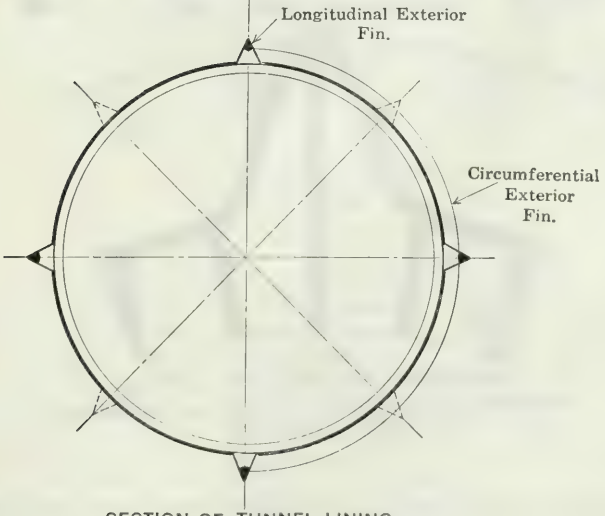
The function and location of such ribs applied to tunnel lining are somewhat analogous to that of bilge-keels on a ship. In effect, however, their range and influence would be greater. Thus, the top and bottom exterior longitudinal fins would resist horizontal pressures or displacements, the horizontal fins (and all others in the central horizontal third) would resist vertical pressures or movements, and the longitudinals would also resist ram pressures and air pressures.

Circumferential exterior ribs, however, by their location and soil anchorage, are especially adapted to resist ram and air pressures; in fact, their resistive action against interior air pressure is similar to that of hoops on a water tank or barrel. The use of inside ribs or tie-rods, etc., to resist air pressure in a tunnel is on a parallel with the use of inside hoops to resist hydraulic pressure in a tank or barrel. The action of air pressure in a cylindrical tunnel is identical with that of hydraulic pressure within a cylindrical tank, and the means used for controlling forces identical in action and effect should be similar. In this case the strategic point of control and attack against distortions and displacements is outside, not inside. Tie-rods and bracing can avail but little, and they are a great inconvenience. The admission of sufficient quantities of soil through the shield doors, as mentioned by the authors, is one means of outside control, in that it relieves outside pressures. Modification of existing practice, as suggested, is another means of outside control of both exterior and interior forces, and this concerns largely the proper design of a lining which will not only sustain permanent and temporary stresses, as the authors state, but also be capable of supplementing skin friction by direct soil support and resistance by anchorage therein, thus warding off and counteracting from the outside forces of distortion and displacement.

Mr. Aylett.



SECTIONAL DETAIL OF LONGITUDINAL EXTERIOR FIN.

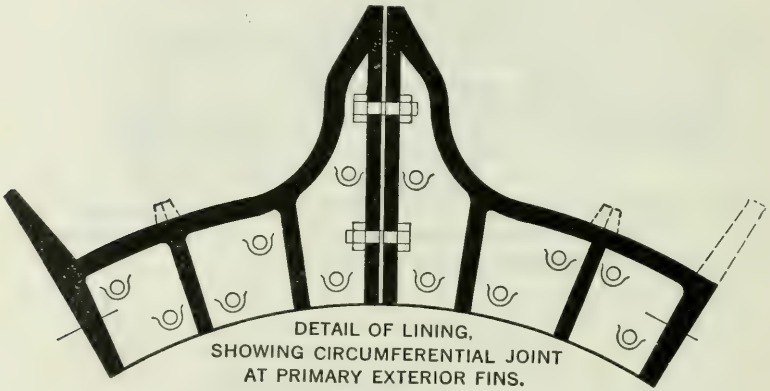


SECTION OF TUNNEL LINING, SHOWING LOCATION OF EXTERIOR FINS.

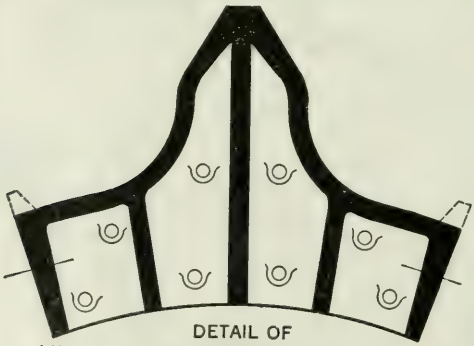
FIG. 19.

Mr. Aylett.

Another advantage afforded by such exterior protruding fins would be their action as cut-off walls for escaping air currents or water. Such currents travel along lines of least resistance, which are undoubtedly between the exterior surface of the lining and the adjacent soil. It is evident that the presence of the exterior longitudinal and circumferential fins in the paths of water or air currents tend to lead them outward, thus somewhat relieving the wet material adjacent to the



DETAIL OF LINING, SHOWING CIRCUMFERENTIAL JOINT AT PRIMARY EXTERIOR FINS.



DETAIL OF LINING-BLOCK FOR BREAKING JOINTS AT EXTERIOR LONGITUDINAL FINS, THUS PRESERVING THEIR CONTINUITY.

Fig. 20.

skin from the usual agitation, churning action, etc., of air pressure through leaks. This action renders such material, a viscous lubricative fluid around the entire circumference. Thus, frictional resistance between the lining and its surrounding soil is maintained to a more or less extent by protruding fins, according to soil conditions, etc. The additional construction sectional area provided by the fins affords

ample abutment space for rams, and thus breakages of lining would be avoided. When necessary and desirable, rams of higher power may be located in line with the main longitudinal fins—and those of smaller capacity intermediate—thus proportioning the ram pressures to their respective supporting abutment areas, the ram stresses being transmitted and distributed to the lining through the longitudinal fins and directly to the soil by circumferential fins. Mr. Aylett.

There has been considerable professional discussion recently as to the merits of metal and concrete lining blocks. Strange to say, a composite block has not as yet been brought forward. The field appears to be both interesting and promising, and is worthy of thought and development. The ideal composite block, of course, would be one that embodied the good qualities of the combination of materials composing it. The blocks shown in Figs. 20 and 21 are designed with a

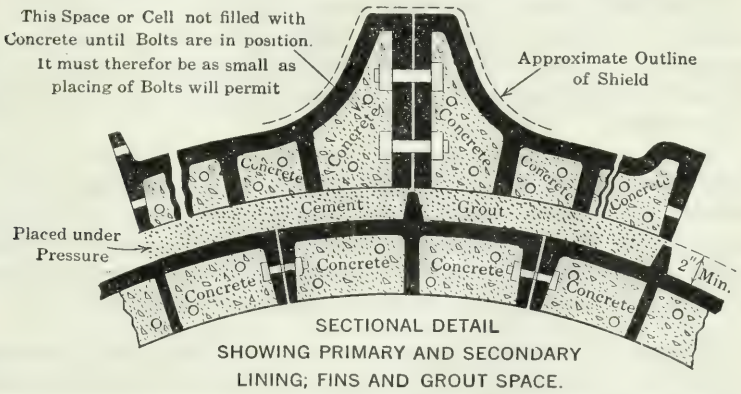

This Space or Cell not filled with Concrete until Bolts are in position. It must therefor be as small as placing of Bolts will permit
Approximate Outline of Shield
Concrete
Cement
Grout
Placed under Pressure
2" Min.
SECTIONAL DETAIL
SHOWING PRIMARY AND SECONDARY
LINING; FINS AND GROUT SPACE.

FIG. 21.

view to use either with or without concrete, as conditions may demand. The advantages of such a block are the greater additional strength and rigidity afforded by its primary and secondary exterior fins. Furthermore, the interior radial flanges and diaphragms divide its convex surface into rectangular voussoir-shaped spaces or cells, so that when filled with concrete the result is a concrete voussoir lining with perfect contact metal joints, the concrete thus performing a dual service as portion of block and as permanent tunnel lining.

The concrete may be placed before or after bringing the blocks into the tunnel. In order to avoid any form work or centering whatever for concrete, all blocks above the horizontal diameter, at least, should be filled prior to being placed in position. Those below may be filled after being placed, as the work proceeds, thus affording a good surface for construction operations. The lighter inner ring shown in Fig. 21

Mr. Aylett.

is intended for use with large diameters only or when soil conditions demand. Such a section would be absolutely water-tight and capable of holding its own in any emergency. In the entire double-ring section, Fig. 21, there are combined, three different linings adaptable to varying conditions and diameters, namely:

1.—A cast-iron or steel block lining provided with exterior longitudinal fins and interior voussoir cells. This section alone is suitable for severe conditions and large diameters.

2.—The same as No. 1, with concrete voussoir lining. This section affords permanent lining without centering.

3.—The same as No. 2, with a secondary composite ring and grout water-proofing cushion between. This cushion is placed under pressure in longitudinal cylindrical sections of the tunnel.

Thus, No. 1 alone, or its combinations with Nos. 2 and 3, provides a means of repeated, rapid shifting from one section to another, as unforeseen conditions may require, without loss of time, leaving a permanent concrete-lined tunnel, as the work proceeds, with any of these combinations.

Blocks of the general character shown may be of cast iron, or structural steel, and with some modification it is also possible to roll a suitable steel section of this general type and of cutting such a section to proper lengths. A pressed steel section is also possible. The grout space will permit of the use of a secondary lining of varying depths with no interference of the interior diameter or clearance.

Mr. Parmley.

WALTER C. PARMLEY,* M. AM. SOC. C. E.—This paper is valuable for the many facts of experience given and for the discussion of principles which are suggestive, although the speaker cannot always agree with the conclusions reached. The authors rightly utilize the Rankine formulas for earth pressures, but apparently fail to take into full account certain limitations or modifications which need to be made when applying them to actual practice. It must not be overlooked that the Rankine formulas are founded on one property of earth and one only—that of internal friction—and they ignore the universal fact that earth in its natural state is more or less compressible and has very material, and, in some cases, determinative, cohesive qualities. Where earth is disturbed during construction its original nature may be altered temporarily and even permanently, so that unequal settlements may take place, thus greatly modifying the direction and intensity of the thrusts. After readjustment it may become either more, or less, dense than it was in its original condition.

* New York City.

Mr. Parmley.

Therefore, the fundamental statement of the authors that

"The horizontal pressure on the tunnel is always at least equal to the active pressure, p_1, but if the condition of internal stresses in the tunnel lining should so require, a greater horizontal pressure will be developed of such magnitude as will be required to provide the necessary equilibrium, but under no circumstances greater than p_2"

the speaker believes to be incorrect.

The late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., many years ago, recorded numerous experiments and citations showing that the active pressure p_1 was rarely equal to, and often less than, one-half the value given by the Rankine formula. The speaker has also made many observations of the pressure of coal, sand, earth sides of excavations as indicated by the deflections of the timbering and the behavior of retaining walls, and in nearly all cases the pressure was less than the value of p_1 given by Rankine. So pronounced and persistent were these facts that more than twenty years ago, when the speaker was designing the Walworth Run Sewer in Cleveland, the active horizontal pressures on the sides of the arches were fixed at one-half only of the value given by the formula and the stability of the arches was determined by this condition.

The second part of the authors' statement just quoted needs some interpretation. In order to establish equilibrium between the internal and the external work it is necessary for the arch to deflect. In deflecting, some parts of the arch will move outward against the earth and others will move inward, or retreat from the earth. It is obvious, therefore, that the abutment property of the earth can do no work in supporting the arch unless the arch tends to move outward against the earth.

In this outward movement the earth becomes more compressed, but compressibility or elasticity is a property which is excluded entirely from the formulas, and the conclusions, therefore, are affected by this limitation.

The outward reaction may be either less or greater than p_2 if the effects of cohesion and compressibility of the earth are taken into account. The amount of work which the earth can perform in giving support to the arch depends on the distance through which it can act, as determined by the actual displacements taking place at the several points along the contour of the arch. To illustrate: Consider a circular tunnel ring, having a radius of 100 units, in fairly uniform, though not hydrostatic, material; it will be deflected to an approximately elliptical form with the short axis vertical. The variations in the horizontal pressure due to increasing depth from top to bottom will not

Mr. Parmley.

be included. The pressures on both the upper and lower halves, therefore, are equal.

In Fig. 22, the full line represents the original circle. The broken line represents the tunnel after deformation has taken place. That is, the vertical radius has become less and the horizontal radius greater so that the deformed shape is that of an ellipse. As the two curves intersect at the 45° points, nearly, a glance at the diagram is sufficient to show that only the lateral quadrants, so far as abutment resistance is concerned, are effective in sustaining the tunnel from spreading, the movements of the upper and the lower quadrants being away from the earth instead of toward it.

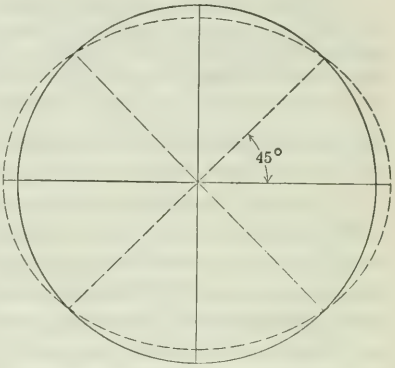


FIG. 22.

The effectiveness of the side resistance is plainly maximum opposite the center line where deformation is greatest, and decreases rapidly both above and below the center becoming nil at the 45° points. The rate in the decrease in effectiveness will be somewhat greater than is indicated by the decreasing deflections, because, unless loaded almost to its limit, equal increments in settlement in most earth imply increasing increments of loads as the earth becomes more heavily loaded and the ground becomes more compressed. However, on the basis of equality of increments of loads and displacements, the rates of change in effectiveness can readily be calculated.

The equation of the original circle is $x^2 + y^2 = 100^2$, and the equation of the ellipse having, say, a 1% change in the original length of the radius, is

$$\frac{x^2}{(101)^2} + \frac{y^2}{(99)^2} = 1.$$

The amounts of compression in the earth may be represented by the increases, Δx, in the values of x for any given value of y as the circle changes to the ellipse.

In Fig. 23 a portion of the side of the arch is shown with the displacements measured in a horizontal direction, the horizontal scale being magnified in order that they may be more easily observed.

The numerical amounts of these displacements for points up or down from the center, are shown in the diagram, being calculated for distances of 10, 20, 30, etc., up to 70, which latter point gives zero displacements at 45° from the center. If the width, 10, of each portion

Mr. Parmley.

shown, is multiplied by its average length between the two curves, that is, a mean value of the Δx, the area obtained will represent the proportion to the whole reaction which each 10-unit vertical height of the ring will be able to sustain. Each of the several percentages is indicated on its appropriate space, the total for each horn-like area above or below the center being, of course, 100, while the actual area is 50.

Within the range of deflections likely to be experienced in an actual case, the percentages for each portion of the whole thrust cut off by the horizontal lines would remain practically constant whether the actual deflection amounted to more or less than the 1% assumed in determining the foregoing figures.

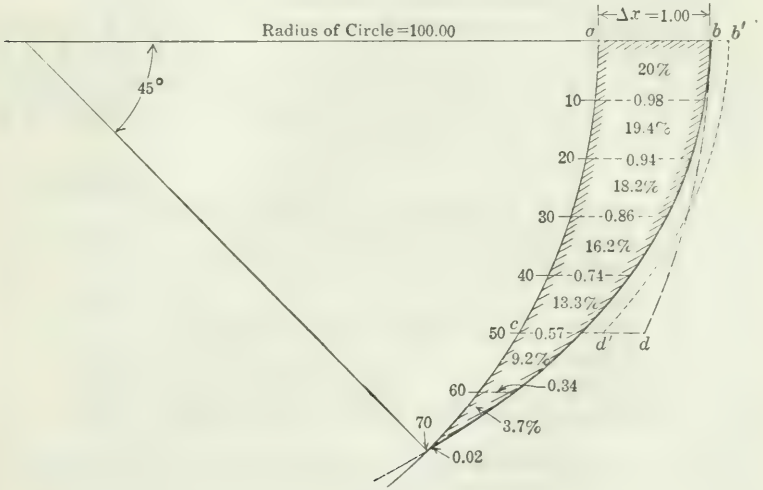


FIG. 23.

If the total effects for each belt of width 10 be added, from the center up to 50, or one-half the radius, it will amount to 87% of the total effect on the full 45° arc. As the 45° point is approached, the horizontal displacements become so small that it is doubtful whether much reliance can be placed in them, since appreciable compression nearly always takes place before the supporting power of the earth is developed to a useful extent. It seems reasonable, therefore, to ignore all horizontal abutment effects beyond one-half the radius either way from the center. This condition requires that the 13% be distributed over the remaining area, or averaged, so that the total area shall remain unchanged. If the pressure is regarded as constant for the remaining width over which it is applied, its intensity will be equal to the maximum at the center. The total reaction then may be shown by the area, *a-b-c-d.*

Mr. Parmley.

It is likely, however, that in fact the actual pressure will not be constant, because the varying amounts of deformation will develop varying amounts of compression. It is perhaps more nearly correct, therefore, to consider that the 13% of the total pressures lying below and above points one-half the radius from the center is distributed over the middle belt uniformly, in which case the total resistance of the abutment would be represented by the area, a-b'-c-d'. In this case, the belt of uniform width, b-$b' = 0.14$, is added to the area remaining after cutting off the tip of the horn below Point 50.

If the ground is hard enough to sustain an abutment load represented by this area, the ring will be sustained with only a small amount of deflection, but if not, serious spreading will take place.

A very useful set of results has been calculated by Mr. B. Haukelid* and from these formulas the following reactions are tabulated, giving the intensity of the reactions at the abutment when spread over various vertical widths, in which p represents the intensity of the pressure and r the radius of the circle:

	Intensity of pressure.
If reaction is concentrated into one horizontal force applied at the center.....	1.12 pr.
If distributed uniformly over one-fourth the diameter	2.42 p
If distributed uniformly over one-half the diameter	1.31 p
If distributed uniformly over the side quadrant.	1.10 p
If distributed uniformly over the full diameter.	1.00 p

The necessary lateral support in terms of the vertical unit of loading thus becomes known for the various depths of abutment shown. As the vertical loads per square foot are usually determined with a measurable degree of confidence, if for a given kind of ground the reaction as shown does not exceed the abutment strength of the ground, the structure will stand.

In order to bring into sharper relief the relationship between vertical pressures and lateral stability needed for different modes of its application, several curves are shown in Fig. 24. The angles of repose are given as abscissas, and the intensity of horizontal pressures and resistances as ordinates, Ordinate 1 being that which represents the intensity of the vertical pressure, p. Utilizing the Rankine formulas for the present, Curve No. 1 shows for any declivity of repose the active horizontal pressure, being the locus of the expression, $\frac{1 - \sin. \phi}{1 + \sin. \phi}$.

* *Engineering Record*, June 14th, 1913.

Mr. Parmley.

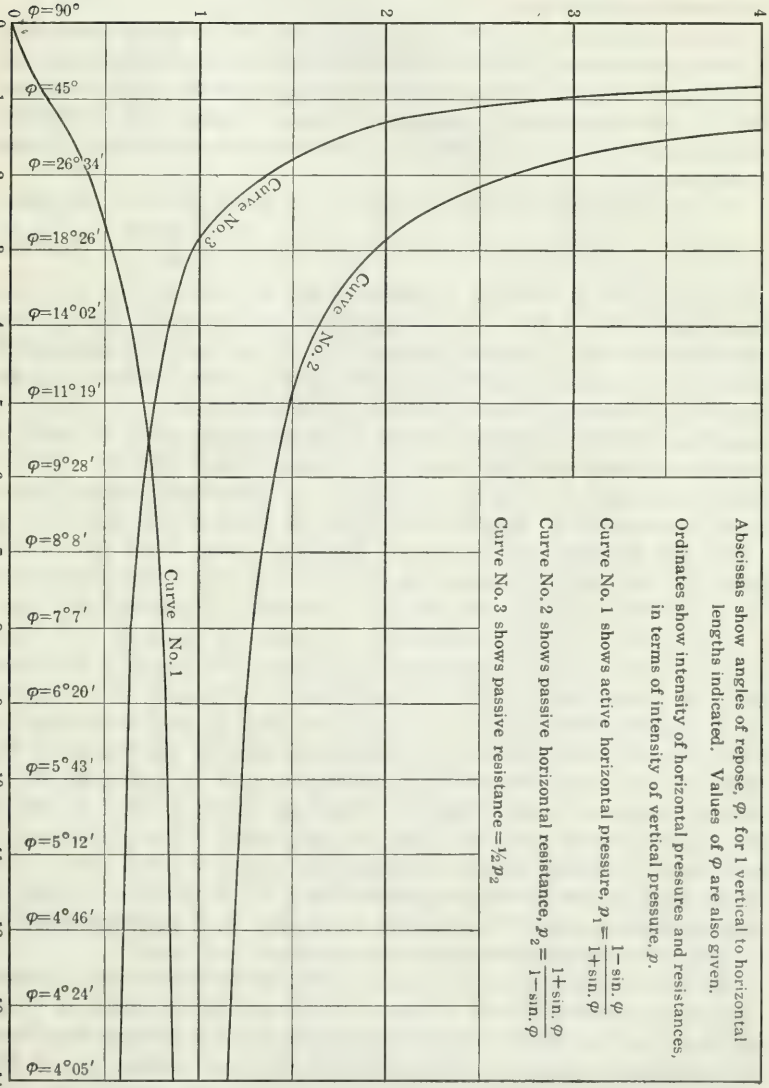


Fig. 24.

Mr. Parmley.

For a vertical cliff, $\phi = 90°$, the pressure is zero, and as the angle decreases the pressures increase until for fluid or hydrostatic case the pressure equals the vertical pressure. Ordinate 1, therefore, is an asymptote to the curve.

By comparisons with the intensities required as given previously, it is at once obvious that the active pressures cannot sustain the outward thrusts from the vertical pressures except for the hydrostatic case. For all others there still remains a lateral thrust which must be absorbed through internal stiffness of the structure, or by the resistance of the earth.

In the diagram, Fig. 24, Curve No. 2 is the locus of Rankine's equation for maximum horizontal stability, $\frac{1 + \sin. \phi}{1 - \sin. \phi}$. Theoretically, for a vertical cliff, the resistance would be infinite, but the resistance rapidly drops away as the angle of repose becomes less and less and the curve finally meets Ordinate $1 = p$, for the hydrostatic condition. If full reliance could be placed on this curve, representing the stability of the actual earth, the ring would be safe even for quite fluid conditions of the ground, providing the structure could be built in intimate contact with the earth. As already stated, however, the fact that the ground must be compressed by an appreciable amount before useful resistance is developed, to rely on the support indicated by Curve No. 2 would be hazardous. The speaker has never felt that he could rely on more than about one-half the values represented by Curve No. 2 and believes that ordinarily it is not wise to go beyond this limit.

Curve No. 3 shows resistances, therefore, of one-half the values given by Curve No. 2. Comparisons of the values given by this curve with the intensities for the various widths of abutment show that the ring would be sustained for a 1 on 2 material if firm contact at the sides extends along a width of at least one-half of the diameter; but that for any softer material the deficiency must be taken up in the internal stresses in the ring. Conclusions, however, must be drawn from such charts cautiously and by the exercise of a great deal of judgment. Cohesion tends to increase the lateral stability, but, on the other hand, materials with a high degree of cohesion are likely to require more compression in order to develop their resisting qualities and so tend to neutralize the beneficial effect of cohesion.

Because of these reasons and the fact that cohesion when present operates to prevent the development of the active pressures, it is not believed that in any but exceptional cases can active horizontal pressures have much effect on the top and the bottom quadrants, and that reliance must be placed chiefly in the resistance against the side quadrants to prevent spreading. The case of ideal active pressures discussed by the authors is interesting and instructive as illustrating

the operation of certain laws, but cannot be applied directly to practice without considerable modification. Mr. Parmley

The speaker agrees with the authors that in a tunnel lined first with cast-iron or concrete segments that the primary lining should be strong enough to withstand all the bending moments caused by construction operations without dependence on further support, not only for the reasons which they give, but because of the practical impossibility of determining just what the ultimate pressures of the earth will be and the lack of constancy of these pressures, due to the changing character of ground penetrated. To feel safe, the engineer should make the structure capable of meeting a greater range of conditions than he believes will actually have to be met.

With the opinion of the authors regarding the suitability of precast concrete blocks for tunnel linings, the speaker does not find himself in agreement. In fact, some of the statements made seem little short of absurd to any one having experience in this kind of construction. The practicability of blocks is not limited to dry soil and favorable conditions, as is abundantly proved by existing important sewers and tunnels built in most difficult situations. Instead of being less able to withstand bending stresses than cast-iron segments, as stated by the authors, the contrary is the fact, as can easily be shown. Thus, assume that the tangential component of the thrust at any given joint cuts that joint at the limit of the middle third, there is then a state of no pressure at one edge and double the average pressure at the opposite edge of the joint. Any further eccentricity of load will produce tension in the ring, and a consequent tendency for the joint to rotate on itself. Take, for example, a cast-iron lined tunnel as now proposed for the vehicular tunnel under the Hudson River. This ring is 14 in. in depth from the outside of the iron to the inside of the stiffening flanges. If the tangential thrust at a given joint, say, at the springing line, is 100 000 lb., which is about what it will be, and if the thrust cuts the joint at a distance of one-sixth of 14 in., or $2\frac{1}{3}$ in. from the center line, the ring will be on the point of rotation or a state of tension at the radial joint. The bending moment, therefore, at the point where actual tension and rotation begins is 100 000 times $2\frac{1}{3}$ in., or 233 000 in-lb. Actually, the moment is less than this because, for simplicity of illustration, the center of the joint is taken as 7 in. from the outside instead of only $4\frac{3}{4}$ in. from the outside to the center of gravity of the section.

A concrete block ring to meet the same situation would be about 33 in. in thickness. If a 100 000-lb. thrust be applied at the limit of the middle third, it will produce a moment of 100 000 times one-sixth of 33 in., or 550 000 in-lb., or about two and one-half times the capacity of the cast-iron joint before tension begins. Nor should it be for-

Mr. Parmley.

gotten that concrete block construction lends itself to reinforcement and that, contrary to the alleged relative weakness of the concrete block structure at the joints, it can be reinforced with steel bars embedded in grooves and grouted in as the rings are erected, thus making the concrete of vastly greater strength.

Contrary to the views of the authors, also, the speaker regards the advantages almost wholly on the side of concrete blocks. Should settlement or distortions cause any cracking or opening of the joints, penetration of the water will do no harm to the structure itself, as the cement grout or mortar of the joint is not affected. On the other hand, should there be any leakage of a finished joint between the cast-iron segments, the bolts of the joints will come into contact with the water and ultimately become weakened by rusting.

The authors apparently fear that blocks of a concrete ring would be liable to displacement because of the small arc of the circle which they might occupy, and their methods of overcoming these imaginary displacements are so fanciful as to suggest that they have not made sufficient study either of the geometry or mechanics of the matter or speak without any basis of experience in this type of construction. Nor need the matter of water-proofing cause worry, as will be appreciated by engineers of experience in this class of work. The fact that many hundred thousand and even millions of dollars worth of work, under all sorts of conditions and difficulties, have been built successfully is sufficient refutation of the specious argument that concrete block construction is not suitable for subaqueous tunnels.

As the proposed vehicular tunnel under the Hudson River is particularly in the minds of engineers in the vicinity of New York City at the present time, and as it leads up to seemingly unnoticed possibilities in the large double-deck roadway tunnel, a property of arches which seems to have been overlooked by engineers, will now be referred to, as no mention of it has been made in the discussions up to the present time.

In Fig. 25, let the horizontal line, *A-X*, represent an axis of abscissas and *A-Y* an axis of ordinates. The points marked 10°, 20°, etc., represent points along a circular arch, 10°, 20°, etc., from the top, and include points down to the springing line at 90 degrees. Bending moments in the arch are plotted upward for positive moments (those producing tension in the inside of the ring) and negative moments are plotted downward, *A-X* being the axis of zero moment.

The curve, *BCD*, represents the moments for a circular ring supporting and sustaining uniformly distributed vertical loads, the moment at 0° being $0.25\ pr^2$, where p is the load per square foot and r is the radius of the circle. At 90°, the moment is the same, but with the negative sign, the point of zero moment being at the 45° point.

The moments for the lower half of the ring will be a repetition of those shown, counting the 0° at the bottom. Mr. Parmley.

If there is a lateral pressure uniformly applied from top to bottom, its effect will be to shorten the several ordinates in the same proportion

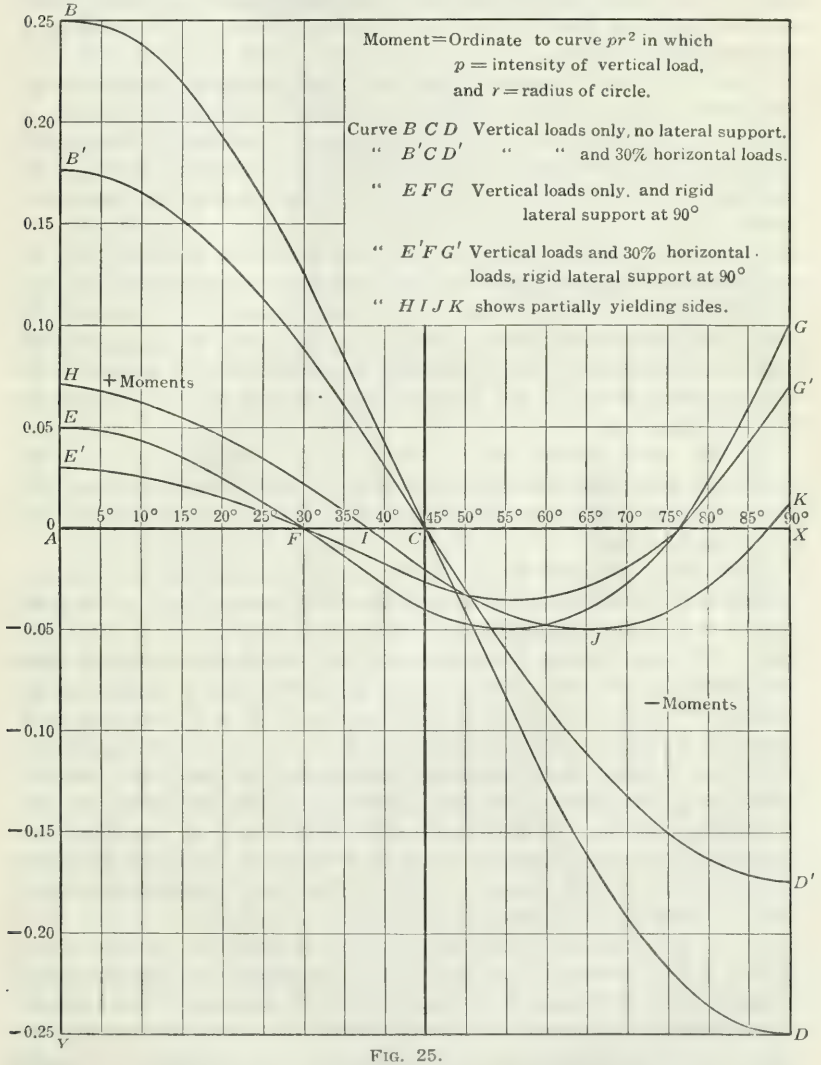


FIG. 25.

as that which it bears to the intensity of the vertical load. The curve, $B'CD'$, represents the case where the lateral pressure intensity is uniform and equal to three-tenths of the vertical intensity. As the

Mr. Parmley

lateral pressures approach the vertical in intensity, the curve of moments approaches the axis, *A-X*, or zero moment, thus producing tangential thrusts only in the ring. Curves *BCD* and *B'CD'* and similar ones are applicable to arches and rings having no lateral abutment supports, so that the arch or ring may deflect freely according to the law of the loading.

Now, consider the case of absolutely rigid abutment support at the 90° point, so that there can be no spreading whatever of the arch or ring. In this case, the moment curve for uniformly distributed vertical loads is shown by the curve, *EFG*, where, at the top, the moment is only 0.049 pr^2, or about 20% of its value where there is no lateral support. In this case, furthermore, there are two points of zero moment, one at about the 30° and the other at about the 75° point from the crown, and the moment at the springing line, 90°, now becomes $+ 0.101\ pr^2$, or about double the moment at the crown. In case there is a uniformly distributed horizontal pressure of 30% of the vertical, the curve, *E'FG'*, represents the moments; the moments for corresponding points in the lower half of the ring are the same as for the points shown.

In the same manner as for the case of no lateral support, as the horizontal pressures approach equality with the vertical pressures, the moment curve approaches the axis, *A-X*, and the hydrostatic condition is again reached.

It must be admitted that these two cases represent the two extremes of possible conditions of support, the first representing no support whatever and the second the case of absolute support at the springing line. All actual cases, therefore, will fall somewhere between these two extremes, and just where a given case will fall is a matter for the judgment of the engineer. For example, if in a given case it is assumed that the condition of support is such that the springing line yields an amount that represents one-quarter of the way from the condition of no support to that of fixed support, the curve for this condition may be considered at each point to have traveled one-quarter of the total journey it would have to go to reach the limit of unyielding support, since the ratio of the rate of travel to the whole distance would be the same for each point.

Such a condition, therefore, may be represented by the curve, *HIJK*. Any other similar curve may be drawn by approximation graphically to represent a given condition of support. The unavoidable errors of judgment as to the character of a given ground and the rigidity of the support are so much greater than the errors that may result from imperfections in the curves that practicable accuracy is attained for any desired condition.

Now, a fundamental law of physics is that for equal pressure in all directions there must be no internal friction, that is, the material must

be a perfect fluid. Conversely, if the material is not fluid there is internal friction and there is an inequality between the vertical and the horizontal components of the weight. Where there is internal friction the passive lateral resistance must be greater than the active horizontal pressure. Rarely is it possible in actual practice to have a case that does not fall somewhere in the intermediate and somewhat indefinite zone between the extreme curves of Fig. 25. Mr. Parmley.

In the recent discussions of the Hudson River Tunnel, these facts, shown in Fig. 25, appear to have been entirely overlooked when considering the nature of the ground that will be penetrated and the means necessary to be used in order to resist the varying pressures that will be encountered.

If a tunnel is adopted, however, which is large enough to accommodate two levels of roadway, one above the other, it becomes possible to realize the extreme limit where fixed supports at the sides are afforded. A horizontal tie or strut can be built into the tunnel construction as it is erected, extending across the horizontal diameter of the tunnel, which will absolutely fix the span of the arch and thus enormously reduce the bending moments, as shown by the curves of Fig. 25.

The lower half of the tunnel becomes an inverted fixed 180° arch, the exact counterpart to the upper half, except for the somewhat increased horizontal pressures due to the greater depth; but the increased depth will not seriously alter the condition of fixed vertical tangents at the ends of the horizontal diameter or of fixed horizontal tangents at the top and the bottom of the tunnel. They will remain nearly constant during all the variations of loading conditions, and this, together with a constant span of arch, produces the condition necessary for fixed 180° arches.

Means for fixing the horizontal diameter of the tunnel can readily become part of the permanent roadway construction for the upper level and, therefore, would not be visible in the finished work. Fig. 26 shows the cross-section of such a possible structure. The internal diameter shown is 38 ft., and the thickness of the wall is 42 in. Without going into detail, the stresses and thrusts in this ring are less than they are for a tunnel having an internal diameter of only 25 ft. and a thickness of 33 in., and they are less than in the proposed cast-iron lined tunnel with an exterior diameter of 29 ft. Furthermore, this ring, with the length of the horizontal diameter fixed, will sustain loads for the entire range of conditions from a material that stands on a natural slope of 1 vertical to 3 horizontal, and the horizontal pressures equal only 52% of the vertical, through the whole range of unstable materials even to the hydrostatic condition, without theoretically even requiring reinforcement. In order to withstand this range of possible conditions,

Mr. Parmley.

a full ring of the same size unsupported at the center by interior construction requires a thickness of 48 in. of concrete heavily reinforced.

Other features which are believed to be important in this plan, but which will be only touched on, are the cores which are designed to slide within the blocks to take the reactions from the jacks and so relieve the blocks from all breaking or shearing strains or possible displacements when the longitudinal joints are staggered. This matter was discussed* at the meeting of the Society on March 19th, 1919. The cores, as shown, consist of high-grade, hooped concrete containing sections of steel I-beams at the center. Cores thus designed of 11 in. in diameter are easily capable of taking the entire thrust of 72 jacks of 200 tons each, the amount of power required to drive a shield of this size, if it is equipped with relatively as much power as has been used in other tunnels built under the Hudson River. If made to meet near the centers of the block-rings, the cores may be utilized later to tie the rings together longitudinally, and when the rings are fully grouted, will give great longitudinal strength to the tunnel.

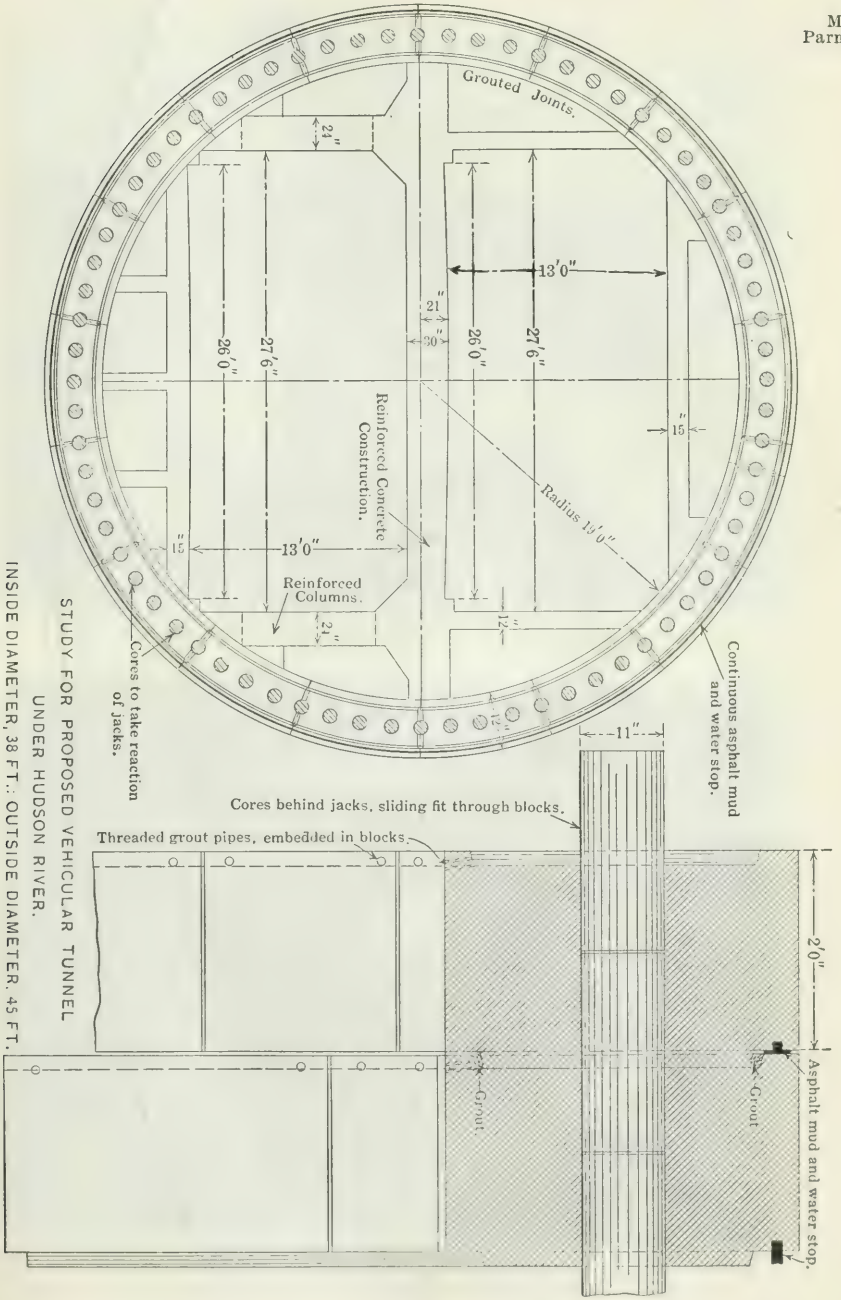
The great advantage in a tunnel of sufficient size to permit three trucks abreast simultaneously in each direction, over that to be gained by the two-line tunnels now contemplated, should justify further careful consideration of the construction of a tunnel of this type.

For a tunnel of large diameter, the possibilities of a structural steel concrete lined tunnel, as indicated by the authors, might well be investigated. The life of the structural members could hardly be expected to equal that of cast iron or concrete, because either Bessemer or open-hearth steel suffers much more rapid corrosion than cast iron, and the concrete suffers none at all.

There is a great disadvantage, however, in any plan requiring a composite ring, such as cast iron or structural steel with concrete lining. Either the ultimate strength of the tunnel will reside exclusively in the strength of the iron ring, the strength of the concrete ring, or in the combined strength of the two materials when acting in the ratio of their respective moduli of elasticity. Figured in this way the composite section cannot be permitted to sustain tension at any point greater than the accepted standards for concrete, unless it is still further reinforced with steel bars. If the ring, on the other hand, is figured as a cast-iron ring acting alone, although of less effective thickness, the fact that the iron, whether wrought or cast, may safely withstand many times the tensile stress permissible in the concrete, results in giving a greater effective strength than is obtained when the two dissimilar materials are figured together. The concrete, unless built very heavy, will add practically nothing to the strength of the

* In connection with the informal discussion of the New York and New Jersey Vehicular Tunnel, p. 459.

Mr. Parmley.



STUDY FOR PROPOSED VEHICULAR TUNNEL UNDER HUDSON RIVER.
INSIDE DIAMETER, 38 FT.; OUTSIDE DIAMETER, 45 FT.

FIG. 26.

Mr. Parmley.

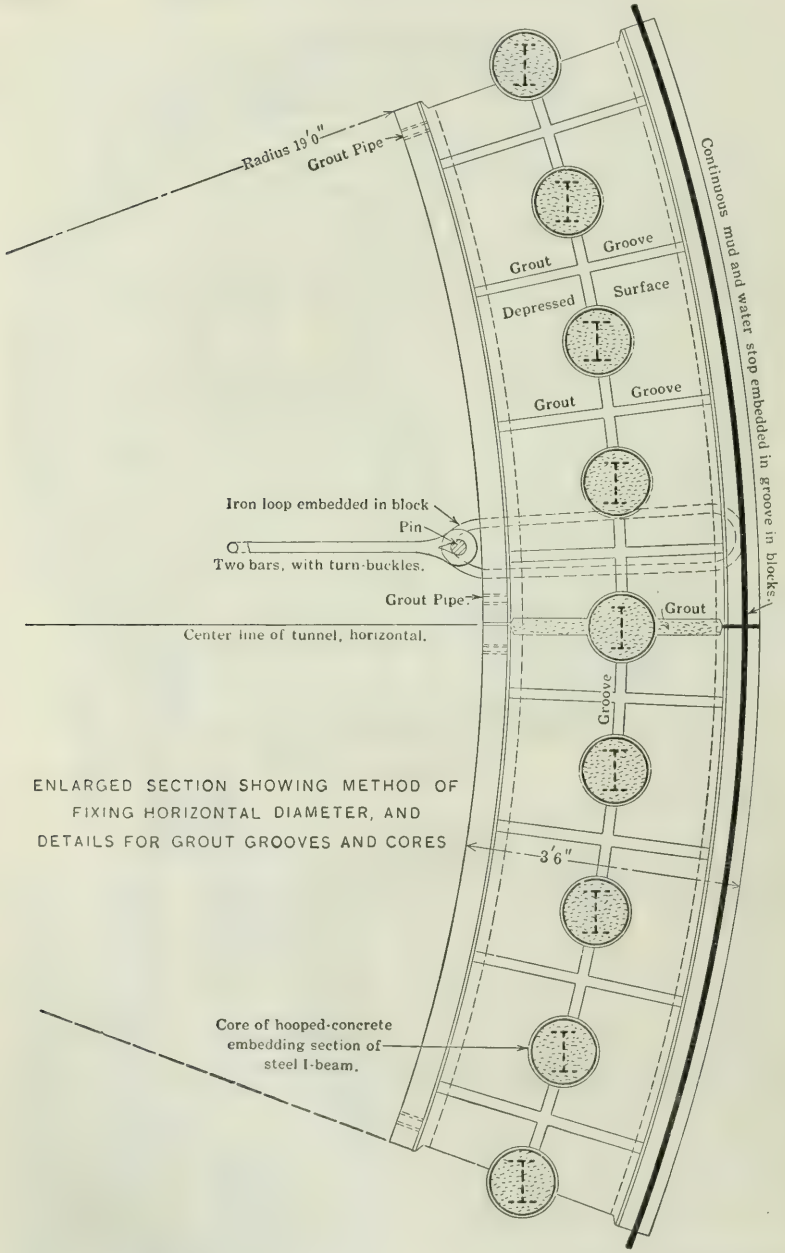


FIG. 27.

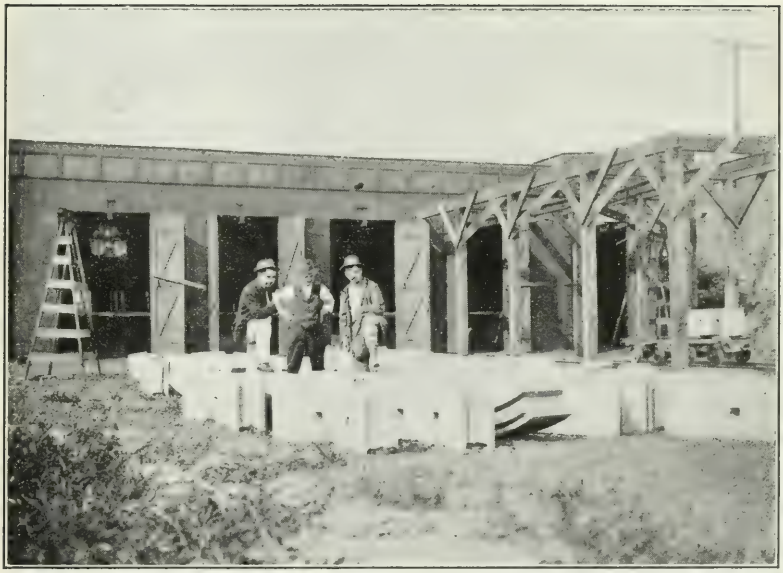


FIG. 28.—PRECAST CONCRETE BLOCKS FOR TUNNEL WORK.

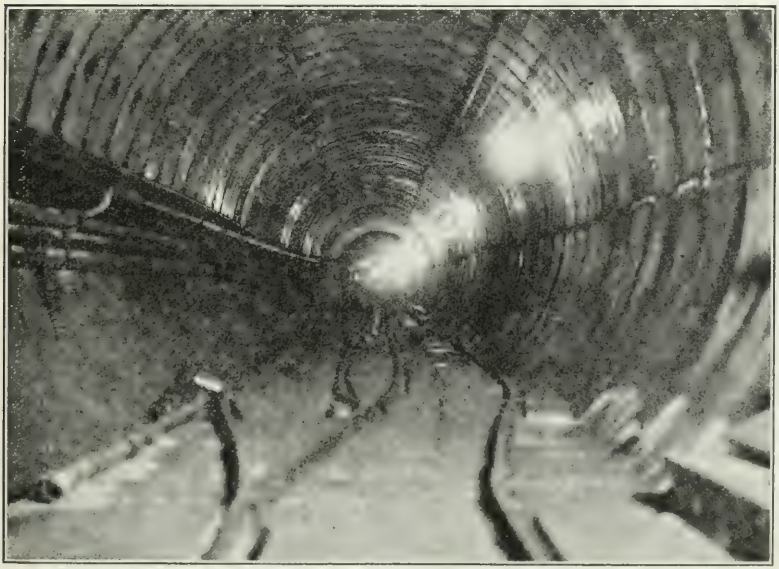


FIG. 29.—INTERIOR VIEW, CLEVELAND WATER SUPPLY TUNNEL.

tunnel beyond greater stability due to the added weight. This fact can be easily verified by calculation. Mr. Parmley.

In contrast to this structural weakness, if either plain or reinforced concrete rings consisting of concrete blocks are used, a homogeneous section of material is obtained, the properties of which are well established. The strength, therefore, may be readily calculated and the results given full confidence.

The concrete blocks being only a substantial fraction of the cost of the cast-iron segment throws the balance of advantages clearly on the side of the concrete structure.

Some have feared that concrete may become disintegrated by the waters under the Hudson River, but past experience in the waters around New York City and vicinity give no proof that any such corrosion of precast concrete blocks would ever take place. The claim does not appear to have merit sufficient to warrant extended discussion.

Any claim that the block tunnel could not be erected, or, if erected, would fall to pieces as it is composed of so-called loose blocks (such claims have actually been made) is too childish to merit serious answer. Fig. 28 shows some of these so-called loose blocks for an actual tunnel (representing more than $1 000 000 worth of them), with the construction of which the speaker was intimately connected. They were erected and they have not yet fallen apart, although the tunnel is deeper below the water surface and much of it is in worse material than will be encountered under the Hudson River. The speaker refers to the intake tunnel under Lake Erie at Cleveland, Ohio. Fig. 29 shows the completed tunnel, which is a standing monument, and will stand for ages, to confute the "doubting Thomases" who are now endeavoring to convince the public that cast-iron segments are the only possible or safe lining for the proposed vehicular tunnel under the Hudson River.

The authors have indeed performed a useful service in giving the Society a paper of such thought-provoking character.

The speaker is glad indeed that Mr. Holland makes a statement as to his open-mindedness toward tunnels of various types of construction, because that open-mindedness will tend to help the construction, whatever type it may be. In regard to the bending moment stresses in the rings, whether they are of cast iron or concrete makes no difference. Mr. Holland's statement assumes that in soft ground there would always be much greater bending stress than in harder, dryer material. The facts are much to the contrary. There is likely to be a much greater bending stress in a structure at the bottom of a ditch than when it is put in a tunnel heading. The heaviest stresses come often at the bottom of a deep ditch, where the material has been disturbed, and where it can have its full effect on the ring; so far as the speaker's analyses have gone—and he has considered various condi-

Mr. Parmley. tions under the Hudson River as applied to these propositions—as the more liquid conditions are approached the bending stresses decrease, while the thrust increases. The thrust causes no trouble, if it can be kept near the center of the ring. It is the bending stresses that cause trouble.

Mr. Holland apparently omitted to note that the concrete blocks are perfectly capable of being reinforced, and in a great amount of work there has been provided solid steel connection between the joints, which he seems to want. That is what the speaker advocates in every case where the transverse bending moments seem to demand it.

As to the question of water leaking in, the study suggested by Figs. 26 and 27 contains many details which were not indicated. It can be shown that by means of an asphalt strip, water pressure can be held back until the ring can be grouted from the inside.

Even without this provision, of the speaker's knowledge it has been found that at 120 ft. below the level of hydrostatic pressure, where the tunnel was in mud, it has been possible to make it tight, whether the air pressure was on or not. The tunnel referred to was 10 ft. in interior diameter; but the mud will come through a $\frac{1}{4}$-in. crack just the same whether the tunnel is 10 ft. or 100 ft. in diameter.

In Fig. 29, showing concrete block construction in this tunnel, long irregular shadows appear, due to the fact that an irregularity amounting to only $\frac{1}{4}$ in. throws a shadow 1 in. or more long, and gives an erroneous impression. Indeed, this concrete tunnel has a splendid interior surface, as far as regularity goes.

In regard to shearing stresses from the jacks, Mr. Holland is entirely right, and it is to overcome them that the speaker suggests using the cores, as shown in Figs. 26 and 27, placed so that they will not be rigidly bound in the blocks, thus relieving them of the shearing stresses, which it is practically impossible to do with cast-iron segments.

Mr. Holland. C. M. Holland,* M. Am. Soc. C. E.—In describing the behavior of tunnel lining during construction, the authors refer to the movement of the Pennsylvania Railroad Tunnels under the North River and the settlement of an East River Tunnel, which the speaker presumes is the Battery Tunnel, although it is not mentioned by name. The speaker believes that the authors do not intend to convey the impression that the deviation from theoretical line and grade was due wholly to weaknesses in the tunnel lining. These deviations were due principally to lack of control of the tunnel shield.

It is stated in the paper that the shortening of the vertical diameter and the lengthening of the horizontal diameter, which is the second and last change in shape of section which the cast-iron lining under-

* New York City.

goes in Hudson River silt, was continuous in the Pennsylvania Tunnels until the secondary lining was placed. The speaker does not understand this to be true from the reports of that work. If this condition was true, it might indicate a much more serious stress in the lining, particularly after distortion had reached certain limits. Of course, this distortion could be taken care of by interior reinforcement which would reduce the elastic deformation of the ring. In any event, there would be, under heavy loading, a considerable elastic deformation in the circular ring similar to that which would occur in any structural member under bending, and the larger the diameter the greater would be the elastic deformation without actually approaching the point of failure. Mr. Holland.

The stresses in a tunnel lining under construction vary greatly with respect to the care taken both in operating the shield and assembling the segments:

1.—Thrust of jacks depends on method of excavating face.
2.—Shearing and bending stresses are often excessive, due to:
  (*a*) Variation in size of segments, causing lips on the circumferential joint.
  (*b*) Variation from circular form, often causing open horizontal joints.
  (*c*) Face of ring not a true plane, causing lips in circumferential joint.
  (*d*) Foreign material between segments.
3.—Binding of the tail on the exterior lining causes heavy, unbalanced loads.

All these matters are of the greatest importance, and, unless careful attention is given to them, will result in stresses far in excess of those due to earth pressure after the lining has left the shield. Most of the cracked plates are broken inside the tail of the shield for the reasons just stated.

The curves of progress given in the paper are of interest, but it appears to the speaker that to a certain degree they may possibly give undue weight to the progress of small tunnels. There is no question but that the progress as shown has actually been attained, but it does not follow necessarily that greatly increased progress cannot be made in tunnels of larger diameter with the present increased experience in tunnel driving. Tunnels recently constructed have been of the smaller type and, consequently, the benefit of experience gained in other tunnels has been of the greatest help and has made possible faster rates of progress.

There is a limit to the size of tunnel which can be driven at all, in that the diameter of the shield affects the amount of cover required overhead. In connection with the driving of the East River Tunnels

Mr. Holland.

of recent date, the rule generally followed was to have a cover one and one-half times that of the diameter of the tunnel whenever the water pressure was balanced at the bottom of the tunnel. This was a ratio worked out from computations based on the weight of the material forming the bed of the river, and it will vary with the character of the material through which tunnels are being driven.

In regard to the caulking of cast iron, it is stated that in order to be effective it must be done under air pressure, and when deformation takes place on the removal of the air, the caulking will be disturbed. The use of rust-joint caulking makes it necessary to caulk in compressed air, but, in recent years, this has been displaced by the use of lead which is always made tight when the tunnel is under normal air just before placing the interior lining of concrete. It would not seem that the movement of these tunnels, effected by tidal changes, would be sufficient to cause leakage, for, from the information in regard to the leakage of tunnels, it appears that, in terms of area exposed to seepage, the North River Pennsylvania Tunnel is one of the dryest tunnels in the vicinity.

In regard to the permanency of steel lining when exposed to salt-water action, there is some question. Reference is made to the shield erected in 1890, abandoned in 1891, re-excavated in 1902 after a lapse of eleven years, and used to complete one tube under the Hudson River. This shield was buried in the mud where there was little, if any, flow of water-bearing fresh acids or rust-producing chemicals continually to the structure itself. However, as the Hudson River silt is practically water-tight, a section of the tunnel in this material, if made tight, might be in very little danger of rusting. In the sand and gravel sections the conditions are plainly different. Articles of wrought iron and steel found during the excavation of the East River Tunnels in the shore sections show marked decomposition. A chain was badly rusted; a hatchet was so disintegrated that it simply furnished the outline of the original tool; an anchor and a small piece of chain showed definite marks of corrosion. It may be well to note in this connection that the prevailing idea is that the flow of ground-water toward the sea keeps the earth saturated in the vicinity of the shore with fresh water, such as is found when wells are sunk along the shore of the ocean. It was found in reading ground-water elevations in Clark Street, Brooklyn, that this elevation was approximately that of mean low water of the East River. This seemed to be out of the ordinary, and an effort was made to find its cause. It was learned that hotels and office buildings were pumping large quantities of ground-water from wells on the premises. This section, being almost wholly covered by pavement and buildings, only a small percentage of the rainfall percolated into the ground. Consequently, these wells were drawing

so much water that the flow was from the East River rather than toward it, and the water pumped was slightly brackish. Shaft caissons with a steel lining have been sunk for tunnel work, but it is believed that in computing the strength of such caissons finally in place that full value should not be allowed for all the metal used. Mr. Holland.

Design.—In regard to the stress diagrams shown in the paper, and from which certain conclusions are reached, it does not appear that, with the loading symmetrical about the vertical axis, the line of pressure should cross the vertical axis at one point only. The speaker has not made any analysis covering the specific cases mentioned, but it appears to him that, under the condition of symmetrical loading about the vertical axis, the line of pressure should intersect the neutral axis at two points.

The eccentricity of the circumferential force in the lining for tunnels, the diameters of which vary from 12 to 36 ft., is stated empirically by the authors to be:

$$e = \frac{d^2 \sqrt{d}}{480}$$

The application of this formula to tunnels which have been analyzed by the speaker's engineering staff for specific loadings, varying somewhat from those stated in the paper, gives an eccentricity for a 29-ft. tunnel of 9½ in., while the analyses give an eccentricity of 2 in., and for a 42-ft. tunnel, the formula gives 23 in., while the analyses give 3 in. This discrepancy may be accounted for somewhat by the difference in loading, and also the fact that, in the analyses, 3° was assumed to be the angle of repose, but it does not appear that this discrepancy can be explained entirely by this variation.

The analyses which have been made in connection with the 29-ft. tunnel which has been recommended to the Bridge and Tunnel Commissions, under conditions of loading very similar to those used by the authors, indicate that the reactive pressures in the surrounding ground become effective when the angle of repose exceeds 2° 20′; whereas, the authors find that a 30-ft. tunnel under the assumed conditions cannot develop the reactive pressures until the angle of repose exceeds 22 degrees.

The speaker emphatically protests against the statement by Mr. Parmley that the New York and New Jersey Bridge and Tunnel Commissions have only under consideration twin cast-iron tubes. The Commissions and their engineers have stated repeatedly that they are open-minded on this question and in their report to the Legislatures of their respective States, they have shown alternative types of construction, using either monolithic concrete or structural steel with an inner lining of concrete and a protective covering of concrete on the outside. Further, the steel lining as discussed by the authors is now

Mr. Holland.

under consideration as well as other types of construction. The preliminary report of the Commissions as to cost was based on twin cast-iron tubes as it was considered that, with the limited time for study, it was the only method of construction which could be definitely recommended. The responsibility for the successful completion of this work rests on the engineers advising the Commissions, and they are in a position very similar to that of trustees in the administration of a trust fund. No trustee would be permitted to make any other than sound, conservative investments with funds under his charge and, similarly, no engineer in spending public funds should proceed with a method of construction which is not absolutely certain of success unless the saving which can be effected would make such a risk justifiable. None of the methods of construction thus far studied has warranted the elimination from consideration of twin cast-iron tubes which have as a precedent the twenty-two tubes already constructed and now in use under the East and Hudson Rivers.

The speaker has never questioned the durability of well placed concrete when built in a monolithic structure. One of his objections to the concrete block tunnel is the thousands of feet of joints which are introduced between the blocks, each of which is liable to leakage. Every engineer who has had to do with the building of concrete structures, particularly in tunnel work, appreciates the difficulty of getting a water-tight joint between two concrete surfaces.

Referring to the stress analyses on which some engineers place so much reliance, and the very complete demonstration submitted by Mr. Parmley, the speaker wishes to warn against the conclusions derived from such a source in that the complicated equations may be mathematically correct but their value depends on the assumptions. Frankly, the speaker considers Mr. Parmley's analytical discussion to be excellent mental gymnastics, dangerous in the extreme if the conclusions are applied to a shield-driven tunnel as the criterion of its adequacy. The speaker does not mean by the preceding wholly to condemn the application of structural mechanics to a tunnel lining for he has made similar analyses of tunnel rings, but they have been used only as one step in their design.

In the design of a tunnel lining to be built by the shield method under the Hudson River, it should be recognized fully that there are other materials which are to be encountered besides Hudson River silt, such as rock, sand, gravel, boulders, etc. All these materials are met with, and the lining must be capable of withstanding the worst set of conditions which can be brought to bear on it, so that a discussion as to whether or not there is cohesion in the Hudson River silt or as to whether Rankine's theory of earth pressures should or should not be applied, is not the real point at issue. In the speaker's

opinion, it is very largely a matter of the application of common sense based on experience, with such theory as is necessary to obtain a rational design. Mr. Holland.

In an old tunnel drift (Fig. 17), that was encountered during the construction of the Borough Hall Station, Brooklyn, N. Y., on the Clark Street Tunnel line, the material varied from fine sand to a coarse sand with gravel and in places there was sufficient cementing material to make it stiff so that it could be dislodged with difficulty under the blow of a pick. This latter condition is that of the material directly over the cap. In this earth, there is a considerable vibration from overhead traffic yet the load on the cap is only a very small portion of the total dead weight. There is a great lack of uniformity in materials and due to this variation serious conditions of unbalanced pressures are certain to arise. In his discussion of this paper, Mr. Parmley has dwelt with considerable emphasis on stresses in the tunnel lining as determined by earth pressures.

The timbering set, as shown in Fig. 17, is a very simple structure, yet the conditions which such a timbering set must meet in tunneling, are very indeterminate. In the construction of the Clark and Montague Street Stations in the tunnel under Brooklyn Heights, it was of the utmost importance to prevent any settlement of the ground, and every effort was made in timbering to wedge it so tightly that it would meet the ground rather than let the ground come down upon the timbering. The sizes and spacing of caps were computed approximately but, in some cases, it was found that intermediate caps were required. In certain places, the ground was unusually heavy due to the presence of water or to the fineness of the sand and the absence of all cementing material. The timbering was maintained successfully for more than a year without any serious disturbance to the buildings due to the fact that levels were taken on the caps and when movement was shown, all wedging which, by the way, was done by steel wedging between steel plates, was tightened and, if necessary, additional bents were inserted.

The study of the behavior of a tunnel lining under an assumed loading is of considerable value, but tunnel engineering is not a science in which the tunnel engineer deals with determinate loads such as is the case with the bridge engineer. It has been the speaker's experience that any tunnel strong enough to be built behind a shield has proved strong enough thereafter. The old Rapid Transit Tunnel from the Battery, in New York City, to Joralemon Street, in Brooklyn, N. Y., is a good example of a tunnel lining which was badly crippled during construction and yet has proved a sound structure ever since its completion.

Mr. Thomson

T. KENNARD THOMSON,* M. AM. SOC. C. E.—The authors are to be congratulated on the valuable and interesting paper they have presented to the Society. In regard to the hydrostatic pressure at the bottom, the speaker thinks that in all caisson and tunnel work it has been found that the air pressure required has been almost exactly equivalent to the hydrostatic head. If the hydrostatic head was increased by the weight of the silt, which is given as from 104 to 110 lb. per cu. ft., there would be an air pressure of from 75 to 80 lb. in these tunnels, which of course would be prohibitive. This question has arisen frequently in designing retaining walls for deep cellars in New York City. Some engineers claim that the pressures are increased by the weight of the sand and material, while others go to the opposite extreme and claim that as the water is only in the voids of the sand, therefore it is not necessary to allow a pressure anything like the full hydrostatic head; but "the proof of the pudding is in the eating thereof", and any one who has worked in compressed air knows that very rarely is more or less air pressure needed than that required by the direct hydrostatic head.

All engineers know, of course, that steel is a much more reliable metal than cast iron, and although it does not require as much weight for the same strength, the cost of the steel will usually be greater than that of the cast iron and, when driving the tunnel through the Hudson River silt, the lightness would be a disadvantage, as the tunnels are usually much too light until the concrete lining is put in place.

It is known, of course, that steel buried in clean silt under water would last forever, but if one side of the steel is in silt and water and the other side facing the tunnel is of varying temperatures and dryness, it is not known that the steel would not corrode, nor can any one be sure that it would not be destroyed by chemicals escaping from power-houses or other sources, or by electrolysis. This was apparently realized by an inventor who about 25 or 30 years ago tried to raise $7 000 000 to build a tunnel under the East River. The gist of this invention, as nearly as the speaker can remember, was that the tunnel lining should consist of double steel shells, having a space of about 15 in. between them, which space was to be kept full of oil.

Some years ago a large steel dam was built in the West, saving thereby more than 10% of what a good masonry dam would have cost, but soon after its completion the dam was destroyed by rust, and it was found that the large hammer at the bottom of the river had been completely destroyed in a few months owing to chemical impurities in the water. This condition should have been ascertained before building the steel dam. A good masonry dam was then built, making the total cost 190% instead of 90 per cent.

* New York City.

Mr. Thomson.

At present we do not know how many different chemicals are dumped into the Hudson River or are liable to be so dumped; therefore, if the speaker were to use a steel shell in that river he would use a lining which would be sufficiently strong by itself, in case the steel should ever be destroyed.

Mr. Brodie.

ORRIN L. BRODIE,* M. AM. SOC. C. E.—In his remarks, Mr. Parmley referred to the effects of cohesion in the consideration of Hudson River silt. The speaker will not attempt to define Hudson River silt, although he has contemplated silt at the shield heading in an old Hudson River tunnel.

Early in his experience, the speaker played the office of nurse, as it were, to one of those old-fashioned, continuous, concrete mixers. It was apparent that the output was anything but satisfactory as far as constancy of desired proportions of concrete ingredients was concerned. Careful observations were had, controlling gears were changed, and other precautions taken, but still this state of dissatisfaction persisted. The only explanation at the time was that the variable, atmospheric humidity affected the cohesion and, therefore, the flow of the dry Portland cement from the feeding hoppers to the worm screw of the mixing mechanism below. No quantitative values of these humidity effects were obtained beyond those directly indicated, but faith in that style of concrete mixer was considerably shaken.

This experience was recalled by Mr. Haines' reference (page 1871) to the coarseness of Portland cement as compared with the silt in the Hudson River. Therefore, if a material like Portland cement is affected, how much more so, a finer material like river silt, directly subjected as it is to varying water content. Its cohesion would have a marked effect, especially with respect to the actual thrusts that would bear on a structure submerged therein. It is the speaker's belief that all engineers having to do with these matters recognize this, and any discussion that will contribute to this interesting subject is most timely.

Throughout this discussion, the expression, "angle of repose", has been used repeatedly and it appears that there is some confusion in the use of this term. The speaker instances Fig. 14, wherein Mr. Meem shows a side slope, in fact, two side slopes, converging downward from the bed of the stream, or surface of the silt, to the top of the submerged tunnel. The line representing this slope was referred to the horizontal by the angle, ϕ, and Mr. Meem, indicating the slope as a plane of slip, referred to it, and the angle mentioned, as the "angle of repose" of the material he was representing. Others have designated the angle of slope at which material had been observed as standing up in its natural or undisturbed state, as in a bank, as the "angle of repose". The point is that there may be a decided difference

* West New Brighton, N. Y.

Mr. Brodie.

between the slope at which a material may be found standing in its natural state and the angle of repose of that same material. Also, in any of the well known treatments of the subject of earth pressures, there is indicated what has been termed the "plane of rupture", which, it is surmised, Mr. Meem and some others had in mind at the time. It is clear, however, that the angle of the plane of rupture with the horizontal is a different quantity.* Precision and accuracy in the use of terms in this discussion, therefore, are urged, as otherwise the net result will be only a confusion of ideas.

The wide variation of opinion voiced as to what is the true angle of repose of the silt would indicate the desirability of strict definition. Rankine's earth-pressure theory, neglecting cohesion as it does, fails, perhaps, under certain conditions, to contribute as precise values as one could wish for thrusts to be encountered; nevertheless, it may be used for comparative purposes, even in such cases of doubt as the determination of actual values. That is, if the angle of repose is considered to be an index of the ratios between principal and conjugate stresses, or in the case of a subaqueous tunnel, between the vertical intensities of loading and the resulting horizontal thrust intensities, extended investigations through a range of such ratios may lead to interesting conclusions.

The use of Rankine's "Theory of Conjugate Stresses" is indicated by the authors. Their assumption, for purposes of calculation, as to vertical pressure at the top of the tunnels is 4 000 lb. per sq. ft., with weight of material of 120 lb. per cu. ft. Their investigations, results of which are depicted in Figs. 4, 5, 6, and 7, presumably with the same assumptions applying, include also consideration of compressed air in the tunnel at 25 lb. per sq. in. and an air pressure balancing the external pressure at the crown of the tunnel.

Numerous investigations of a similar character have been carried on by the speaker, so this portion of the authors' paper is of great interest to him. He has considered circular tunnels, both of cast-iron and concrete, of various sizes, and also reinforced concrete tunnels approaching the ellipse in shape. Rankine's theory was used and emphasis was laid on the comparative value rather than on any absolute values of the results of these investigations, although the assumptions of conditions adhered to actualities as far as they could be attained, considering the purposes for which the investigations were undertaken. The maximum pressure per square foot at the top of the tunnel, due to 38 ft. of water superimposed on from 25 to 31 ft. of silt, was 5 656 lb. This followed from 64 lb. for water and 104 lb. for silt, per cu. ft. Air pressure was taken at from 37 to 40 lb. per sq. in., depending on the size of the tunnel. For the cast-iron tunnels, three cases were

* The speaker has since noted that Mr. Meem has corrected Fig. 14 accordingly.

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investigated, namely, (I) cast-iron ring, alone; (II) cast-iron ring with compressed air, inside; (III) complete tunnel structure. Cases (I) and (III) were under atmospheric pressure only. (The 25 ft. of silt was used only for the largest circular tunnel.) The cast-iron ring only was assumed to withstand stress. The investigations covered a range in angles of repose from 3 to 20 degrees.

The method of least work and the method of deflections (the latter checking the former most precisely) were developed and used in obtaining the true funicular polygons. It is pertinent to observe here that the old theory, commonly known as the theory of the masonry voussoir arch, applied in the past to investigations of this character, is not correctly applicable to the investigations of subaqueous tunnels or similar structures. It should also be noted that this question is independent of the question of what loadings may be expected to come on the structure. It may be shown that the older method leads to indeterminate results, whereas the methods applied by the speaker in this work always yield results that are completely determinate. The older method is approximate at best, and the results therefrom were found to differ appreciably from those obtained by the more correct methods, especially for low angles of repose where greater precision is necessary, although, in some cases, approximate agreement was had.

Figs. 4, 5, and 6 present striking peculiarities that compel attention. It will be noted, as mentioned by Mr. Holland,* that the funicular polygons, or "stress diagrams", in passing from crown to invert, intersect in each case the neutral axis of the tunnel ring at only one point. This inexplicable behavior of the funicular polygon has never been evidenced in the speaker's investigations referred to. With the application of the compressed air condition (Case II), for an angle of repose of 3°, the resulting polygon has been found to intersect the neutral axis in three points. For higher angles of repose, where the horizontal conjugate thrusts are lower in value than for the lower angles, intersections in four points have resulted; but never in one point, as the authors show. Considering this one-point intersection in the half ring of the tunnel, in connection with the points of application of the crown and invert thrusts, respectively, within the neutral axis at the crown and without at the invert, leads to the conclusion that, instead of being in a state of equilibrium, there is taking place an upward translation of the entire tunnel. This point, admitted by the authors, is dismissed immediately by them with the words, "the soil at the top may resist this tendency of upward displacement"; so it would appear that their analyses are caught "on the wing", so to speak. The subject of dynamics would enter here; yet, a brief consideration of these polygons, from the standpoint of horizontal crown deflections (the resultant

* See p. 1901.

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deflection being equal to zero for equilibrium to obtain), will show that, if they meet the condition of equalization of positive and negative moments with respect to the neutral axis, their one-point intersection will result in a residual, horizontal deflection at the crown, which is a condition that is contrary to the state of symmetry assumed for the entire tunnel ring and its loadings, as evidenced by the presentation in the diagrams of a half tunnel ring.

The authors' Conclusion No. 6, that "the pressure of the compressed air in the tunnel during construction produces bending stresses in the lining", together with other statements of the same nature contained in their paper, is apt to be misleading. These statements as to the effect of air pressure are far from true. If atmospheric pressure at the surface of the river was assumed to be transmitted as a uniform, vertical intensity of force to the river bed, and thence through the silt both vertically and, as a conjugate pressure, horizontally to the tunnel, in other words, if the atmospheric pressure were assumed to act as a surcharge on the silt without the tunnel, while radial within, some bending moment would result. This condition would have to be taken into account in every investigation, whether atmospheric pressure or compressed air only was being considered in the tunnel. On the other hand, however, it would have a constant effect on all stresses, in any given case.

For the assumption of radial air pressure within and without the tunnel ring, the compressed air does not produce bending stresses in the tunnel ring, as no bending moment is introduced by the air pressure. The bending moments induced by the other loadings remain unchanged, whether the tunnel ring is subjected to ordinary atmospheric or compressed air pressure. The authors have not stated, however, how they have assumed the normal air pressure to be transmitted to the tunnel.

It is obvious that the thrusts in the tunnel ring are directly modified by the internal, compressed air, and as the bending moments remain constant, the eccentricities of these thrusts vary widely from those under the condition of atmospheric pressure only. The further statement of the authors, in this respect, that "the eccentricities of the stress [thrusts] increase with increased air pressure," is quite correct.

The following conclusions have been drawn by the speaker from his investigations, principally having to do with two sizes of cast-iron tunnels and along the lines previously stated by him (covering Cases (I), (II), and (III)):

Although the eccentricities in Case (II) (compressed air) varied widely, almost erratically, it might be said, as compared with the

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other two cases, the resulting tensile and compressive stresses in the cast iron varied continuously with the angle of repose and paralleled the similar variation of stresses of the other two cases.

All conditions being considered, the highest values of tension and compression in the cast iron resulted for Case (III) (complete structure, under atmospheric pressure).

Therefore, in order to consider the effect of different conditions of loading on the structure, not only from bending moments, but, in conjunction with them, the thrusts resulting must be treated.

An expression for the maximum eccentricity of the thrust line in the tunnel ring is presented by the authors. It was derived, they state, from funicular polygons of tunnels of various diameters under specific, rigorous conditions. The speaker was led to compare the results obtained by this formula with his own results, by doubt as to the reliability of the authors' polygons as published. It is presumed that the compressed air condition was not included in the formula; but the speaker includes his three cases in Table 2, together with values which he has computed by means of the authors' formula, realizing, of course, that its upper limit is for a diameter of 36 ft. Outside, round numbers are given for $\phi = 3°$ (the formula is based on $\phi = 2° 52'$), the other conditions are respectively as previously stated. It was developed by the speaker's investigations that, except for compressed air, the eccentricities increased continuously with the angle of repose. Table 2 shows the results, in inches, for a 23-ft. and 29-ft., cast-iron, and a 42-ft. (outside diameters), concrete, block tunnel.

TABLE 2.—Maximum Eccentricities of Ring Thrusts.

Size and kind of tunnel	23-ft., cast-iron.	29-ft., cast-iron.	42-ft., concrete block.
Air pressure, in pounds per square inch.	37	40	.....
Ring alone, in inches	½	1	.....
Complete structure, in inches	2	2	3
Ring with compressed air, in inches	4.5	26	.....
By authors' formula, in inches	5.3	9.5	23.7

The discrepancies appearing in Table 2 are too great to be accounted for by the difference in assumed weights of silt of 120 and 104 lb. per cu. ft., but rather indicate that the funicular polygons on which the formula was based were not the true polygons for the cases considered.

The conception of the elliptic arch for use in ordinary earths (where the depth of tunnel ring may be neglected with respect to the depth of the overlying earth) and the hydrostatic arch in water, coupled with

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his investigations of a form of the latter type subjected to silt loads, causes the speaker to express doubt as to the advisability of successfully utilizing the ellipse as a means of reducing bending stresses in the rings of under-river tunnels, especially when the practical features involving difficulties with shield-driven tunnels are considered. Unless necessary in some types, the speaker deems it unwise to depart from the circular shape, for, in some cases, although the bending moments in the upper half were found to be improved, the flatter, lower portion usually exhibited an increase in values of bending moments and the funicular polygons in most cases tended toward the circular shape, generally speaking. This necessitated heavy reinforcement, locally, as both the thrusts and eccentricities were considerable.

The authors have outlined a method for determining the values of critical angles of repose for tunnels of different sizes, under which angles the "abutting power" of the silt may not be used to attain a state of direct compressive stress in the ring. An allusion is made to the method of least work; but nothing further is mentioned as to its use in the paper. It is stated also that coincidence of the "stress polygon" (line of pressure) with the neutral axis of the tunnel lining will give the least internal work.

In this connection, the speaker submits Fig. 30, showing variations of bending moments with the angle of repose (expressed also as the ratio mentioned previously) for the three tunnels already considered and for the complete structures, in each case; and Fig. 31, showing corresponding variation of the ring thrusts for the 29-ft. tunnel, as typical of the others. The bending moments at the haunch are of opposite sign to their corresponding moments at the crown and invert; but for convenience all are plotted in Fig. 30 as of like sign. By means of Fig. 30, the horizontal force necessary to produce the coincidence of the line of pressure, or what is the same thing, zero bending moment, in the ring, may be directly approximated. The diagram shows at once, however, that there is no point at which zero bending moment exists throughout the ring; that is, flexural stresses would occur at some point or other. On the other hand, for small angles of repose, these stresses are negligible, as tensile stress of flexure is more than neutralized by the direct compression. Figs. 30 and 31 give values plotted for crown, haunch, and invert, and the average of the angles where those of Fig. 30 cross the zero bending moment line are shown in Table 3, against corresponding values obtained by the authors for tunnels nearest in size to those considered by the speaker. (The values for the ring alone (Case I) were also considered but did not exceed the values for the complete structures by more than 2 degrees.) Below the angles of repose as tabulated, then, the tunnel rings will not be in pure compression and, above

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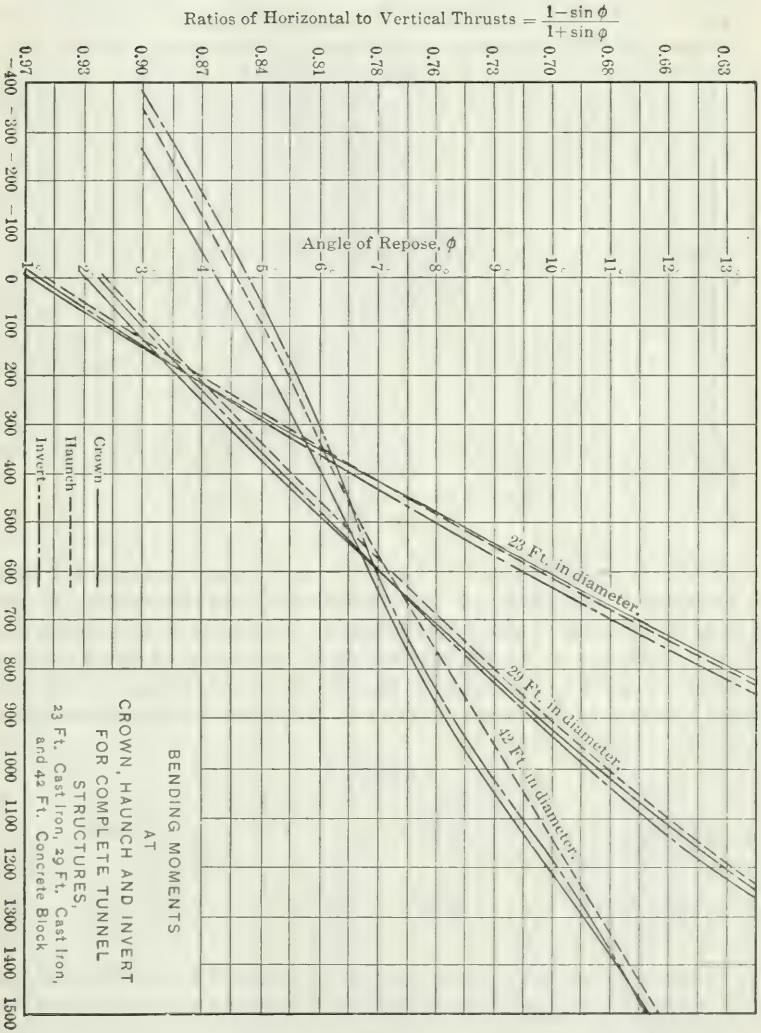
Ratios of Horizontal to Vertical Thrusts = $\frac{1-\sin\phi}{1+\sin\phi}$
0.93 0.90 0.87 0.84 0.81 0.78 0.76 0.73 0.70 0.68 0.66 0.63
Angle of Repose, ϕ
1° 2° 3° 4° 5° 6° 7° 8° 9° 10° 11° 12° 13°
−400 −300 −200 −100 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500
Bending Moments, in 1 000 Inch-Pounds per Linear Foot of Tunnel.
23 Ft. in diameter.
29 Ft. in diameter.
42 Ft. in diameter.
BENDING MOMENTS AT CROWN, HAUNCH AND INVERT FOR COMPLETE TUNNEL STRUCTURES.
23 Ft. Cast Iron, 29 Ft. Cast Iron, and 42 Ft. Concrete Block
Crown
Haunch
Invert

FIG. 30.

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these angles, the abutting power of the silt may be relied on to insure direct compression with the bending stresses neutralized, provided the abutting power is greater than the thrust intensities necessary to cause pure compression in the ring, according to the interesting principle which the authors have enunciated.

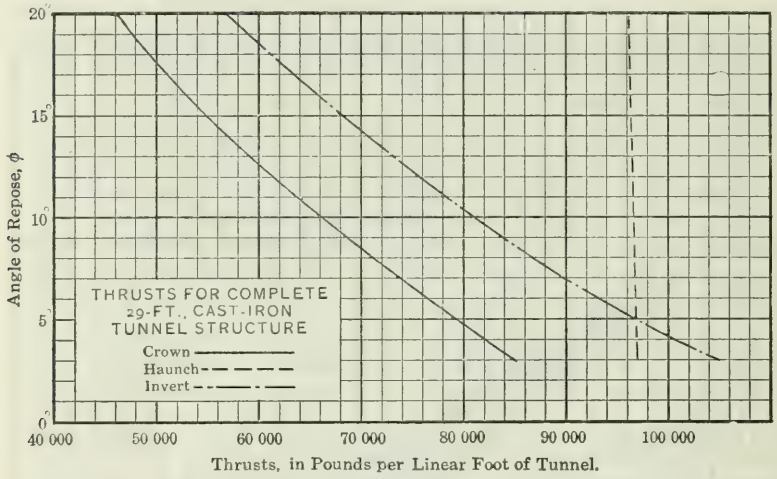

FIG. 31.

Again, it is to be remarked that these discrepancies are too large to be charged against the differences assumed for the weights, as previously stated, and would indicate that their treatment was incomplete.

As the thrusts affect the internal work of resistance, the variations shown in Fig. 31 would indicate that the authors' statement in regard to the position of the stress polygon to give least work may bear some revision.

TABLE 3.

Tunnel	Authors'	24 ft.		30 ft.		36 ft.	
	Speaker's		23 ft.		29 ft.		42 ft.
Critical angle of repose		18°30′	1°10′	22°0′	2°15′	25°30′	4°30′

In conclusion, stress analyses for structures like subaqueous tunnels, are more or less involved and much depends on the assumptions, so that the values quoted may be merely illustrative, although perhaps applicable only to a particular case. Comparative studies, however, if sufficiently comprehensive and complete, and based on sound mechanical principles, are of value in aiding the judgment. It is to be hoped,

Mr. Brodie.

therefore, that the authors will succeed in reconciling the grave discrepancies indicated; for the results as published in the paper may be misinterpreted by and mislead those interested but unfamiliar with the details of such subject-matter.

Messrs. Johannesson and Hewett.

S. Johannesson* and B. H. M. Hewett,* Members, Am. Soc. C. E. (by letter).—When this paper was first drafted, it was intended to cover all kinds of ground, but subsequent revisions removed nearly all references to ground other than that with a low angle of repose, and, therefore, in order to avoid possible misunderstanding, it has been suggested that in its final form the title of the paper should be changed to "Notes on Tunnel Linings for Soft Ground".

As to Mr. Meem's discussion, the writers desire to state that it is not believed by them that the North River silt in its natural state is "soupy", but that its character approaches that of a viscous fluid. The writers' view may be illustrated by the following: Suppose a number of steel billets with smooth dry surfaces are stacked as shown in Fig. 32. They would not slide down, and the force, *P*, required to move them up hill would be great. If, however, the surfaces between the billets were well oiled, the billets would slide down under their own weight, and the force, *P*, required to push them up hill would be much less than before. There is just as much difference between a stiff dry clay and silt. In dry clay the particles rest on one another without a lubricant; in silt, the surfaces are lubricated with water.

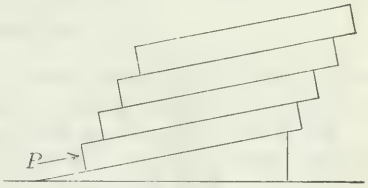

Fig. 32.

The correctness of this last statement is indicated by the fact that the water content of the silt is greater than the voids that would exist if the particles touched each other. This does not mean that the particles are floating in water, but that each particle is covered with a film of water in a state of surface tension. That this is so is supported by the fact that an apparently stiff and dry lump of silt, immediately after being excavated, when rubbed between the hands, disintegrates and shows free water. Incidentally, the tidal rise and fall of the silt may be explained by a change of the thickness of these films of water. The effect of air pressure on the silt is similar; in other words, it decreases the water content of the silt in the near vicinity, removes thereby part or all of the lubricant, and makes the silt less likely to slide.

In his discussion of stresses, Mr. Meem considers only the direct, and not the bending stresses, although the latter certainly have a decided effect on the structure; but even in the method of determining

* New York City.

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the direct stresses, the writers cannot follow him, because by his method, the greater the angle of repose, the larger the load to be carried. Going to the extreme, this would mean that in solid rock the tunnel would have to carry all the rock above, while the greatest arch action would occur in water. Similarly, the formula on page 1862, giving the value of side pressures, would give infinitely large side pressures in rock and no side pressure in water. As regards the top pressures, the writers agree with Mr. Meem that often a reduction may be made to allow for cohesion, but how much of a reduction? In soils with a small angle of repose and where the cover is shallow, the reduction would not be worth taking into consideration.

It might be thought, from the objections made by Mr. Meem, that the pressures as determined by the writers were much greater than those determined by Mr. Meem. It is instructive, therefore, to compare them. Using the numerical values given by Mr. Meem on page 1864, for the formulas for P_2 and P_3 on page 1863, it is found that

$$P_2 = 125\,325 \text{ lb. per lin. ft.}$$
$$P_3 = 187\,500 \text{ " " " "}$$

while the corresponding value given by the writers is 153 750 lb. per lin. ft., or about 2% less than the mean of P_2 and P_3.

Mr. Parmley questions the value of the side pressure used by the writers, stating that it has been found to be less than that assumed. The writers agree with Mr. Parmley that cohesion, etc., probably act as a tendency to produce an active horizontal side pressure less than that assumed. Here, again, the question is: How much allowance may be made? In soils with a small angle of repose this allowance must be quite small, and it is safer not to count on it.

Mr. Parmley assumes that the vertical diameter will be decreased under the earth pressures. Suppose, to go to the extreme, that the tunnel is in water with the top at the level of the surface of the water, then the horizontal diameter of the tunnel would evidently have a tendency to decrease. Similar conditions may occur in a soil with a small angle of repose. It is necessary, therefore, in this case, to consider properly the active horizontal pressure.

Mr. Parmley makes a valuable suggestion regarding the relative deflection of the tunnel structure and the compression of the surrounding soil. This relation naturally depends on the stiffness of the structure and on the character of the soil.

The development of a design for a concrete block tunnel with a horizontal tie or strut, as described by Mr. Parmley, is most interesting. The writers, however, question the advisability, from an operating standpoint, of a tunnel having two roadways, one above the other, except in case of absolute necessity, and they do not believe that the

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cost of concrete blocks erected is only a substantial fraction of the cost of cast-iron segments erected, except in dry soil and in tunnels of moderate size.

The writers acknowledge the valuable work performed by Mr. Brodie in determining stresses in tunnel linings and the accurate mathematical methods used by him. They also acknowledge an error in their paper, pointed out by Mr. Brodie, relative to the stress diagrams shown in Figs. 4, 5, and 6. By omitting the effect of the vertical forces on the horizontal reactions the curves intersect the neutral axis only once. Making this correction, it is found that the curve intersects the neutral axis twice and that the eccentricities are reduced. Notwithstanding this reduction of the eccentricities, their magnitude still remains, under the conditions discussed in the paper, a function of $d^2 \sqrt{d}$, so that the principle of the argument developed is unaffected.

Mr. Brodie states that the compressed air does not produce bending stresses in the tunnel ring, as no bending moment is introduced by the air pressure. The air pressure, however, decreases the direct stresses and, therefore, intensifies the effect of the bending moments. Mr. Brodie further states that it was developed by his investigations that (except for compressed air) the eccentricities increase continuously with the angle of repose. The writers believe that this extraordinary result which, when carried to the extreme, would mean that the lining in rock would have the most severe bending stresses, is arrived at by taking only the active horizontal pressures into account. The fact that the active pressures are considered in action indicates that Mr. Brodie has assumed that contact exists between the lining and the soil. If this contact exists, as has also been assumed by the writers, the passive pressures cannot fail to develop if called on, and they will completely change the bending moments. In Table 2, Mr. Brodie shows a comparison of eccentricities as determined by the writers' formula and as determined by him in specific cases. The writers do not know the details of the assumptions made by Mr. Brodie, but believe that the discrepancies are accounted for partly by a difference in the assumptions as to the weight of the lining and partly by a difference in the assumptions as to the weight of the silt. The writers used, for the purpose of comparison, a constant weight per linear foot of circumference of tunnel ring, while Mr. Brodie probably used the actual weights of tunnel lining, which increase rapidly with the size of the tunnel. Investigations indicate that at a small angle of repose the weight of the lining has a large effect on the bending moments.

With respect to the use of a tunnel lining in the form of an ellipse, the writers do not fear any practical difficulties; in fact, the elliptic form has been used successfully for shield-driven tunnels. As far as

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the theory is concerned, it may be interesting to note that after their investigations had been made, they found that Professor Rankine had long ago arrived at the same conclusion.

In Table 3, Mr. Brodie gives a comparison of the values obtained by the writers for angles of repose above which no bending stresses will occur and angles of repose, determined by Mr. Brodie, giving the least bending moments in the lining. The writers submit that the two sets of values, as determined by Mr. Brodie and by them, do not represent the same matter. Mr. Brodie's values represent those angles of repose which would produce the least bending moments, providing active horizontal pressures only were acting. As just stated, the writers cannot admit such a condition as existing. As stated in the paper, the writers' values are arrived at by taking into consideration the passive horizontal pressures and represent the angles of repose at which no bending moments will occur.

The writers venture, in closing this discussion, to make a few observations on the reasons which led them to offer this paper to the Society and to point out some of the difficulties which surround this subject, which presents an almost virgin field. There is need for much further study and experimentation, and until that work has been done, honest differences of opinion as to the stresses in the lining of a tunnel under any given set of conditions are to be expected.

The art of the tunnel builder is an ancient and honorable one, and goes back to the earliest dawn of man's existence as a tool-using animal. Like all other arts, its development during the last half century has been rapid. In comparison, however, with many other branches of engineering, the development of what might be called the scientific side of tunnel design has not kept pace with practical application. This is perhaps partly, or even largely, due to the fact that among engineering works tunnels are comparatively rare. For every tunnel there are hundreds of bridges, and, consequently, the science of bridge building has kept better pace with the necessity of the art than that of tunnel building.

In old days tunnels were placed in ground which would give easy construction conditions, and where the ground was too difficult for the skill available at that time no attempt was made to build a tunnel. This cannot always be done nowadays, and it is often the case that a tunnel has to be built between two points regardless altogether of the geological conditions which lie between them. Where the ground is adverse, either by reason of carrying large quantities of water, though itself sound and self-sustaining, or where it is of the kind known as "soft" or "heavy", great engineering skill may be required, both in design and execution, to put the work through.

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It is essential to sound engineering to have as thorough a knowledge of the ground as possible, including the magnitude and distribution of the pressures which the ground will exert on the tunnel lining during the various stages of construction up to its finished and operating condition. The effect of such pressures on the lining, as well as the action of compressed air where such is used, must also be known.

Tunnel engineers of experience and prominence, both here and abroad, have suggested, from time to time, that it is useless to attempt to calculate the stresses in a tunnel lining because the earth pressures are not known. As these pressures must be assumed, they say, the proportions and dimensions of the lining may be assumed just as well and the design evolved by judgment and experience. This, no doubt, in many or most instances is the literal truth. It need not be taken, however, as the final word. What we do not know we may find out by experiment and theory founded on the experiment, just as all our knowledge of natural phenomena has been gained.

In the paper the writers have used the theory of earth pressures enunciated by Professor W. J. M. Rankine as the basis of the discussion. It has been objected that this theory is based on the existence of certain ideal conditions which are not found in Nature and that cohesion, the quantity of water contained, etc., would affect the adaptability of this theory to the subject at hand. To this it is replied that, according to such experimental work as has been done, it has been proved that, as far as the Hudson River silt is concerned, when a low value (in this case an average of about 3°), was assigned to the angle of repose, it was found that the pressures calculated by Rankine's formula agreed with those actually observed on the tunnel lining. This suggests the value of a comprehensive series of full-scale experiments on earth pressures on tunnel lining. As long as engineers have no additional information, the Rankine theory affords a working basis for discussion of soils of this character.

A structure must carry the most severe loading that may be brought upon it. If it can be determined, with certainty, that the cohesion, for example, will reduce the earth pressure, an adjustment may be made. If a possible variation of the water content in the ground can cause a variation in the characteristics of that ground, that content which produces the most severe loading must be taken into account.

With regard to the scientific or rational design of tunnels, the most urgent necessity, at the present time, is experimental work to determine the actual earth pressures on as many existing tunnels and on as many tunnels under construction as possible, and to determine the physical characteristics of as many kinds of ground as possible. In this manner important economies of design may be discovered, and this economy may, and doubtless would, repay many times over the cost

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of the necessary experimental and office work. The recent Progress Report* of the Special Committee on Stresses in Railroad Track furnishes descriptions of methods of measuring pressures in the ballast, which are highly suggestive as to methods which might be applied to tunnels. The writers venture to suggest that the Society might do a great and valuable work by an investigation of earth pressures.

The writers cannot close this discussion without an expression of their indebtedness to the labors of the late Brig.-Gen. C. W. Raymond, M. Am. Soc. C. E., during the time he acted as Chairman of the Board of Engineers of the Pennsylvania Railroad Company's tunnels in New York City. These labors lasted from 1905 to 1910 and comprised many hundreds of the most careful and precise measurements and observations made with sensitive and specially designed gauges on the Hudson River silt. These observations, and the conclusions drawn therefrom, which have never been published, confirmed the expectation which had been formed that the silt would behave in accordance with the Rankine theory. This series of experiments was the actual source and origin of the writers' paper.

The Rankine theory had been used by S. H. Woodard, M. Am. Soc. C. E., in a report on the expected action of the silt on a tunnel driven through it, which was submitted in 1903 to the Board of Engineers of the Pennsylvania Tunnels, before the actual construction work was started and, therefore, before Gen. Raymond's experimental work, which determined the actual characteristics of the silt, had been done.

The design of tunnel linings, especially as concerns linings for shield-driven tunnels in soft ground, has been touched on by M. Raynald Legouez, by Paul A. N. Seurot, M. Am. Soc. C. E., and by others, but there seemed to the writers to be a need for the endeavor to find a way by which practical results could be drawn from analytical discussion. The writers recognize that the paper only touches the fringes of the subject, and they feel most gratified that so many able and experienced engineers have filled in, by their discussion, the gaps and deficiencies in the paper. They hope that this important and interesting subject will be pursued further, in the light of wider experimental knowledge, by such means as will develop information regarding the actual stresses in a tunnel lining, so that the art of tunneling may thereby be advanced.

* See p. 1409.

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Paper No. 1460

REVISION OF THE NIAGARA RAILWAY ARCH BRIDGE*

By Charles Evan Fowler, M. Am. Soc. C. E.

With Discussion by Messrs. O. H. Ammann, C. Bentham, J. A. L. Waddell, Theodore Belzner, F. E. Schmitt, P. G. Lang, Jr., James E. Howard, Clyde T. Morris, Almon H. Fuller, and Charles Evan Fowler.

Synopsis.

The revision of the Niagara Railway Arch Bridge made in 1918-19, as here described, comprised first a complete investigation of live loads for railway bridges, in order to determine the probable maximum loading for such a terminal bridge—a double-track, two-hinged, 550-ft. arch over the Niagara Gorge. In 1918 an extensive series of strain-gauge readings, the methods of which are fully described, was made. The dead load stresses were investigated by jacking apart one top chord of the arch span by means of a special toggle, the center top chord dead load stress being read with the strain gauge. This resulted in the jacking open of both top chords in 1919 to change the center shims and readjust the stresses in all members of the arch trusses.

The investigations as to the proper unit stresses to adopt and the logical column formula to use for the revision, are given at some length. The strain-gauge investigation also determined the secondary stresses, and readings of stresses made during the passage of trains at various speeds were made the basis of a formula for a curve of values for impact at various speeds. The new Fuller-West strain gauge, developed as a result of the investigations, is described.

* Presented at the meeting of June 2d, 1920.

The methods of reinforcing the sway and floor system of the arch span and of the approaches, particularly the methods of relieving dead load stress and of doing the field work, and the construction of a copper-steel salt-water protecting floor are described, also the method of grade rectification.

The method of reinforcement of an abutment 64 years old is given in some detail, as well as armored coatings of the arch skewbacks to protect them from weathering, and of skewback thrust walls to carry the sway stresses to bed-rock. Brief descriptions are given of the improvements made to the bridge entrances, and other bridge property accessories. The paper as a whole covers the work of revising the structure up to modern capacity at less than one-third the cost of a new structure.

The Niagara Railway Arch Bridge (Fig. 1) was constructed in 1896-97 from the plans of the late L. L. Buck, M. Am. Soc. C. E., to take the place of the Roebling Railway Suspension Bridge, and has already been described for the Society.* The history of the various bridges which had occupied this site is quite fully given, so that no description of them will be repeated here; but it is desirable for the full understanding of this paper to show briefly the evolution which took place from the first suspension bridge built in 1847-48.

The first bridge was built of wire cables, with no stiffening trusses, carrying a single 7-ft. roadway. That no disaster occurred due to the heavy winds up and down the Gorge may be considered miraculous, as such a disaster did befall the Wheeling Suspension Bridge, which was built by the same engineer, Charles Ellet, whose fame resulted from his work in the development of long-span suspension bridges. Doubtless the study of the faults of this very primitive bridge over the Niagara Gorge led the late John A. Roebling, M. Am. Soc. C. E., to plan the railway suspension bridge with heavy stiffening trusses, and with guys to the cliffs as well.

That there was much competition between the suspension type and the tubular continuous girder, is evidenced by the remark in Mr. Roebling's final report to the effect that "as regards the success of your work more has been accomplished than was promised. The idea of a perfectly rigid structure, such as a tubular bridge, was never held out".

* *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 125.

FIG. 1.—NIAGARA RAILWAY ARCH BRIDGE.

The design for a tubular continuous girder bridge for this site was made in 1850, by James Hodges. This is shown in Fig. 2, and is undoubtedly one of the most pleasing, artistic and chaste designs ever made for a bridge with wholly rectangular lines. One is also led to wonder if it would not, with some reinforcement, have been in use nearly up to the present, had it been built instead of the suspension type. The reason for its not having been adopted, as may be inferred from Mr. Roebling's remark just quoted, was due almost entirely to the question of cost. There may also be some question as to whether such heavy stone piers would long have stood so close to the water's edge, especially in view of the fact that the side slopes under water have scoured out as much as 30 ft. near this site, during the last twenty years.

The statement may be safely made, however, that if an engineer were considering the various phases of the problem of bridging the Gorge at this site to-day, the quest would soon narrow to the three-span continuous truss and a two-hinged arch such as is now in use, with the modification, however, of its being constructed as a three-hinged arch under dead load. The latter feature governed very largely the course of the writer's investigations during the summer and fall of 1918, looking toward increasing the carrying capacity of the arch structure.

The stiffened suspension bridge would now only be considered for railway use where the span was very long, or where the ratio of dead load to live load would approximate 1.8 to 1 up to 3 to 1. There is, however, every probability that as steel becomes increasingly costly, the minimum span for suspension bridges for carrying railways will again gradually approach 1 000 ft.

The process of evolution at Niagara shows also the progress from wood to stone and steel, and, later, to reinforced concrete and steel. The towers of the first suspension bridge were of wood, and although an advance was made in using stone towers for the railway suspension bridge in 1856, a still further advance was made by replacing the stone towers, which had become badly cracked and shattered, with steel ones in 1886. The use of stiffening trusses in the railway suspension bridge was a distinct advance, but they were of wood, and were later replaced with steel trusses. Both the towers and stiffening trusses were replaced under the supervision of Mr. L. L. Buck, and fully

described for the Society.* The work carried out during 1919 on the arch, under the writer's direction, marked a still further replacement of wood with steel, and the substitution of steel or reinforced concrete for some of the stonework.

The writer was first called on in June, 1918, to make merely the regular annual inspection of the bridge, and it was supposed to be confined to the need for ordinary repairs, such as riveting and painting. However, there was found to be very serious corrosion of the top flanges of the floor-beams, stringers, and stringer laterals on the spans, and the top flanges and laterals of the six approach girder spans, from the salt water dripping from refrigerator cars and from leakage of water and injector discharge from locomotives. The studies made in developing plans for repairing this damage, where metal originally ½ in. in thickness was corroded in some cases to a thickness of only $\frac{1}{16}$ in., soon demonstrated that the loads passing over the double-track railway deck were from 30 to 70% heavier than those for which the structure was designed.

Instructions were thereupon given to proceed with a thorough analytical study and strain-gauge analysis of the entire bridge. This was considered to be necessary by the writer on account of the 550-ft. arch span being an indeterminate type of structure; on account of the large secondary stresses that it was apparent were induced from many of the details; and from a desire to fix definitely the amount of impact. The first problem, therefore, was to study the past and probable future increase of live loads.

Live Loads.

The trains crossing the Suspension Bridge at the time of its completion in 1855 were so light as to be somewhat amusing in comparison with present-day trains, but they were a serious problem to the engineer of that period. In his final report, Mr. Roebling speaks of passenger trains of 15-ton cars frequently passing over the bridge, and the average number of trips in March, 1855, as being more than 30 daily, probably including freight trains as well as passenger trains.

The loads used by Mr. Roebling in testing the Suspension Bridge are referred to in his report in several paragraphs as follows:

* *Transactions*, Am. Soc. C. E., Vol. X (1881), p. 195.

FIG. 2.—NIAGARA CONTINUOUS TUBULAR DESIGN.
JAMES HODGES, 1850.

"An engine and tender of 34 tons weight, together with one passenger car, crowded with persons, making a total of about 47 tons, caused a depression in the center of 5½ inches." * * *

"A single engine of 23 tons weight, including tender, caused a flattening of the camber in the center of 0.3 feet." * * *

"On the last mentioned day (March 18th, 1855) the railroad floor was opened for business by passing an experimental freight train, composed of 20 full loaded cars, pushed by a 26-ton engine, from the Canada to the New York depot. The gross weight was estimated at 326 tons."

This indicates that the total weight of each car, with load, was only 15 tons. Other tests were made with a 22-ton American engine and a 34-ton, six-driver, English built, freight engine. The locomotives used in the test trains by Mr. L. L. Buck, in testing the Arch Bridge on July 29th, 1897, weighed 90 tons each, the cars 52.5 tons each, and the total test load on the 550-ft. arch span was 1 670 short tons. The engines used by the writer for load tests and deflection tests on August 30th and 31st, 1918, weighed from 177 to 221 tons each, and the test load on the 550-ft. arch span was 2 490 short tons.

The diagram, Fig. 3, shows by the lower curve the average increase in the total weight of locomotives in short tons for this locality from 1850 to 1918. This curve was established from the weights of locomotives passing over the Niagara bridges during this period and filled in by locomotives used by the writer in calculating bridge stresses in the same territory from 1887 to 1918. It agrees quite closely with the one established by J. E. Greiner, M. Am. Soc. C. E., from the records of the Baltimore and Ohio Railway. The curve extended to 1940 indicates that the total weight of locomotives by that time might reach 365 tons, if the same rate of increase is maintained; but owing to mechanical limitations well known to locomotive builders, such as lack of space for much larger cylinders, it seems more probable that the 1940 limit for consolidation or decapod locomotives will not exceed 300 tons, as indicated by the dotted reverse curve. The total weight of the various Cooper locomotives has been located on the curve, as a means of general comparison. The investigation developed the fact, which is doubtless more or less appreciated by all railway and bridge engineers, that the Cooper loadings are out of date and should no longer be used for calculating bridge stresses. The Cooper E-65

locomotive (and all others) is only 56 ft. in length, whereas the Grand Trunk Mikado locomotives used in the tests of 1918 on the arch weighed 221 tons, or practically the same as the E-65, but were 77 ft. long over all.

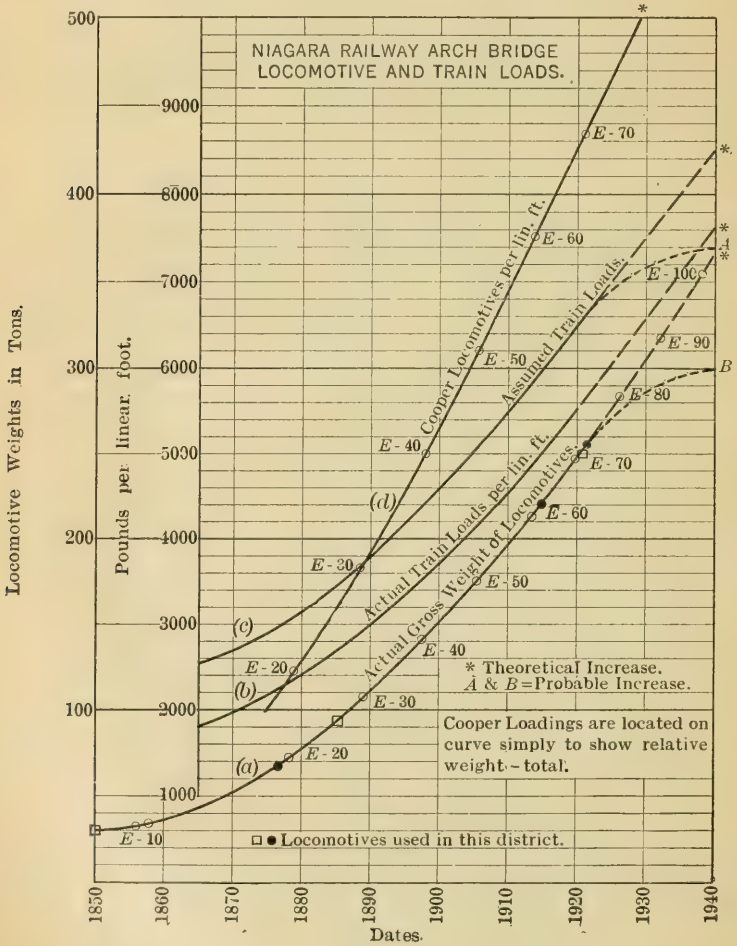


FIG. 3.

The peculiar use of the Niagara Arch as a terminal bridge, or connection between the Canadian and American yards, makes it unnecessary to provide for as heavy locomotives as would be used on a main-line structure, so that Cooper's E-60 as adopted for the arch

and the truss spans reaching to the shores, will undoubtedly be the maximum absolutely required up to 1940; but to make a structure of uniform strength for modern locomotives it was necessary to use Cooper's E-70 loading for the stringers and floor-beams on the spans and the approach girders, or on spans from 20 to 52 ft. in length. Then, by stopping traffic at any time on one track, it will be possible to pass heavy loads or locomotives slowly over the other track, up to about E-80 loading or its equivalent.

The logical procedure for new bridges, however, seems to be to abandon the Cooper loading entirely and adopt a table of moments (and one of end shears to correspond) similar to that given in Column 10 (F) of Table 1. This was established by taking the maximum moment from Columns 1, 2, 3, and 4 for each span length, Columns 1, 2, 3, and 4 being the moments from the heaviest existing locomotives and trains, as calculated by H. T. Welty, M. Am. Soc. C. E., Structural Engineer of the New York Central Railroad. This indicates that for a logical application of the old Cooper loadings, E-70 should be used for spans from 10 to 30 ft., E-80 for spans from 30 to 100 ft., and E-60 for spans from 100 to 300 ft. and over. Should it be desired to provide for future increase, as is advisable, a percentage must be added to the values in Column 9 (F'), logically 25% at the present time, these values plotted, a smooth curve established comprising them, and the moments tabulated for each span length from 10 to 300 ft.

The values for moments in Column (10) (F) of Table 1, and Fig. 4, would be greater than for E-90 loading up to about a span length of 120 ft., and for spans greater than 120 ft., the F moments would be less than for E-90 loading. Some of the large railway lines are already planning to use a similar scheme, and it is confidently expected that some such method will be in universal use within a few years.

The adoption of a maximum loading such as this, and approximating E-90, calls of course, for a more logical system of unit stresses, and not less than those used in the Niagara Arch revision, such as are explained in a later section of this paper. They agree well with the unit stresses used for the Hell Gate Arch and the Quebec Cantilever. The table would show the relation to the Cooper loads, and they could still be used as a reference standard for spans calculated for them.

The actual train loads for the same period are shown on Fig. 3, as Curve (b), and the assumed train loads as Curve (c). These curves

TABLE 1.—Proposed Composite F Moments for Maximum Railway Bridge Loading.

Moments, in Thousands of Foot-Pounds.

Span, in feet.	Virginian. 1 Locomotive, 2-10-10-2 (1)	Erie S. F. 2 Locomotives. 2-10-2 (2)	Mallet. 1 Locomotive. 2-8-8-2 B. (3)	P. R. R. S. F. 2 Locomotives. 2-10-2 (4)	E-60 (5)	E-70 (6)	E-80 (7)	E-90 (8)	F' (9)	F (10)
10	85	90*	85	.....	84	98†	112	127	90	135
12	121	122*	115	.....	120	140†	160	180	122	220
15	189	197*	184	.....	188	219†	250	281	197	330
20	312	323	302	359*	310	361†	413	465	359	570
30	679	714*	610	.....	616	718†	821	924	714	1 150
40	1 185	1 140	971	1 243*	983	1 147	1 311†	1 475	1 243	1 850
60	2 279	2 165	1 810	2 326*	1 949	2 274	2 599†	2 924	2 326	3 350
80	3 937*	3 450	2 940	3 750	3 241	3 781	4 321†	4 862	3 937	5 250
100	5 949*	5 050	4 300	5 350	4 830	5 635	6 440†	7 245	5 949	7 500
150	11 750*	10 300	8 957	10 450	10 585†	12 349	14 114	15 878	11 750	15 000
200	18 130*	17 270	15 500	17 814	17 785†	20 749	23 714	26 678	18 130	24 500
300	34 500	35 600	32 021	37 600*	37 000†	43 167	49 334	55 500	37 600	47 500

* Maximum of eight heaviest trains.
† Nearest of E Class.

F' = maximum each train.
F = composite F' + 25%.

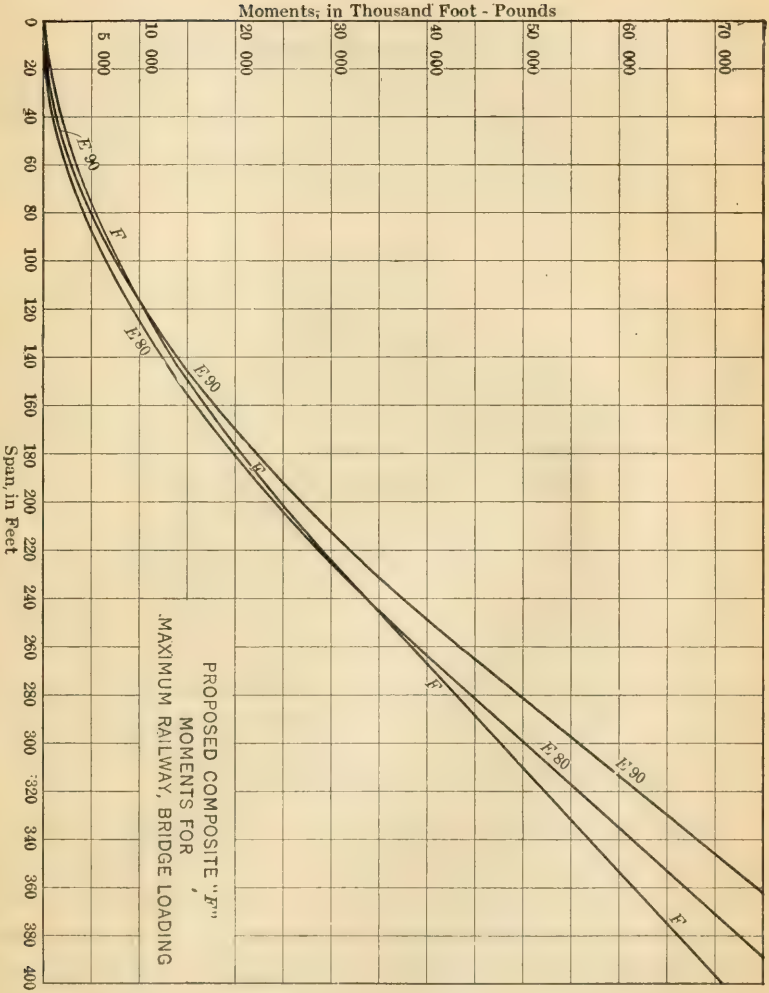


FIG. 4.—COMPARISON OF COMPOSITE AND COOPER LOADINGS.

indicate that the following, or train loads, with each of the Cooper loadings E-60, E-70, and E-80, are too high and that if Cooper's loading is to be used for the future, the train load for E-60 should be 5 500 lb. per lin. ft., for E-70 only 6 000 lb. per lin. ft., and for E-80 only 6 500 lb. per lin. ft. The weight per linear foot of Cooper locomotives is shown in Curve (*d*) for the purpose of general comparison.

The careful study of all traffic using the Niagara Arch Bridge, and especially of that passing over it during the months that the tests were in progress in 1918, indicated that, for both tracks loaded, 90% of E-60 should be used on the arch, but 100% on the trusses of 115-ft. span.

Deflection.

The deflection of the Roebling Suspension Bridge was very large, amounting to 5½ in. under a light load of 47 tons total for one locomotive and one passenger car. This always necessitated the slow speed of 5 miles per hour, and as the trains increased in weight greatly by 1896, the deflection became alarming, amounting to about 18 in. The Schneider Cantilever Bridge near it had a deflection under the original test load of about $7\frac{5}{16}$ in. The new arch built in 1896-97 was tested, therefore, only for the deflection under the test load of alternating engines and cars weighing a total of 1 670 tons, and a maximum vertical deflection of $\frac{13}{16}$ in. resulted. The calculated deflection for the full load on both decks of 10 000 lb. per lin. ft. was $1\frac{5}{16}$ in. This was for a total load on the arch span of 2 750 tons, while the load of 2 490 tons used by the writer on August 30th and 31st, 1918, caused a maximum deflection of 1½ in., or an increase in effective loading of 84.5% above the 1897 test load used by Mr. Buck.

The sidewise deflection of the Suspension Bridge was so excessive as to require the use of stay cables to the cliffs, and although under the proportionately light loads of 1897 the arch was comparatively stiff sidewise, the loads had increased so greatly by 1918, with speeds allowed by sufferance up to 25 or 30 miles per hour, that the side sway had become somewhat alarming in amount. The writer, therefore, reduced the maximum speed to 10 miles per hour in July, 1918, and new sway bracing was introduced, as will be described later, that has practically removed all sidewise deflection.

Impact.

When the inspection of 1918 was undertaken the impact on the structure was apparently very excessive, and was composed of various factors, such as direct impact from the excessive speeds, hammering on open rail joints, bad track alignment, flat wheels, and various kinds of vibration. The impact readings made with strain gauges therefore covered all elements of train load outside the merely static effect. The readings were made with the Berry strain gauge and the curves of increase due to speeds of 5, 10, 15, 20, and 25 miles per hour are shown by solid lines in Fig. 5. These readings indicated that probably the critical speed for the structure was in the neighborhood of 25 miles per hour, at which speed the impact practically agreed with that derived from the old Schneider formula:

$$I = \frac{300}{l + 300}.$$

However, as no readings were made at higher speeds, it can only be stated that at 25 miles per hour the impact agreed with that derived from the Schneider formula; but it is by no means certain that at some point of higher speed values as great as those given by the Lindenthal formula would not be reached.

The values of impact are given in Table 5, which is the stress sheet for the structure as rebuilt. The impact stresses have been computed by three different formulas—the Schneider formula, the Lindenthal formula, and the proposed new American Railway Engineering Association formula; and also for the values adopted for the structure (column headed Niagara in Table 5), as based on the strain-gauge readings, which are for 17 miles per hour, or practically 50% of the Schneider values. The formulas referred to are as follows:

Schneider, $$I_1 = \frac{300}{l + 300}$$

Lindenthal, $$I_2 = \frac{L^2}{D + L} \times \frac{8\,Q + q\,a}{4\,Q + 4\,q\,a}$$

New A. R. E. A., $$I_3 = \frac{300}{300 + \frac{l^2}{100}}$$

Fowler, $$I_4 = \left(\frac{I^2}{s^2}\right) S^2$$

$$I_1$$

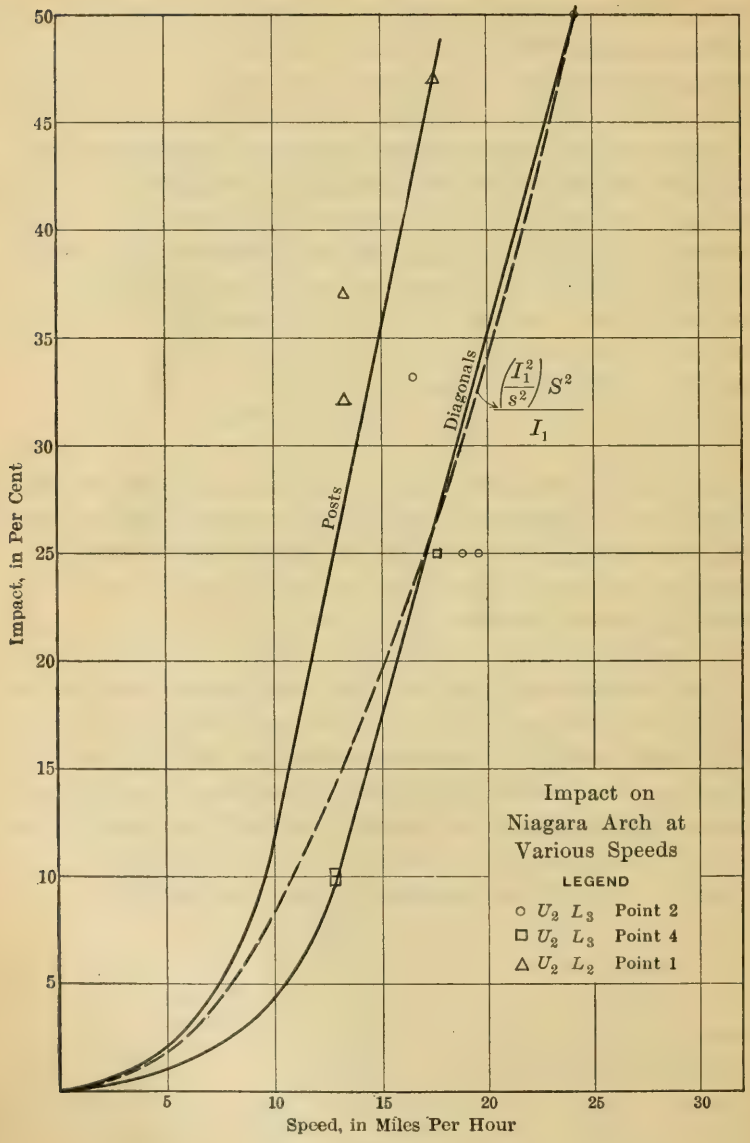

Impact, in Per Cent
50
45
40
35
30
25
20
15
10
5
5
10
15
20
25
30
Speed, in Miles Per Hour
Posts
Diagonals
$\frac{\left(\frac{I_1^2}{s^2}\right)S^2}{I_1}$
Impact on Niagara Arch at Various Speeds
LEGEND
○ U_2 L_3 Point 2
□ U_2 L_3 Point 4
△ U_2 L_2 Point 1

FIG. 5.

in which,

I_4 = percentage of impact for any speed below that read;
L = live load stress;
D = dead load stress;
l = loaded length;
S = speed, in miles per hour, for reduction;
s = speed, in miles per hour, for strain-gauge reading;
Q = locomotive weight;
q = train load per linear foot;
a = length of train behind tender.

Although the readings made by the writer were not great in number, they were so harmonious as to make it possible to establish a safe curve for the variation of impact due to variations in speed below the critical point or that speed giving maximum impact. This curve is shown in the dotted line in Fig. 5 and the formula is that given as I_4 with the denominator I_1, and for which may be substituted the values derived from either I_2 or I_3 if it is desired to use another than the Schneider formula for obtaining maximum impact.

When used for some other structure, check readings should be made for the particular case at several speeds, and the coefficient of S definitely fixed. The actual running speed over the structure should be established of course at some maximum limit, somewhat below the speed used for fixing impact values for the stress sheet.

Whatever impact formula is used, it should certainly be one which gives values high enough to cover as well all the factors of ignorance as to stresses, and thus permit confidence in the logical use of proper maximum unit stresses. With such an idea in view, the impact values given by the Lindenthal formula for the arch do not seem to be very excessive for high speed.

The question of impact from the highway live load was carefully considered, and owing to the fact that the highway deck carries no street cars, and that the maximum highway live load is small in proportion to the train load adopted, no impact allowance was made for this part of the moving load. When still further additions are made to the structure and street cars provided for, then an impact allowance will be made on the street car live load, and also for any additional future loading that may be of a highly dynamic character.

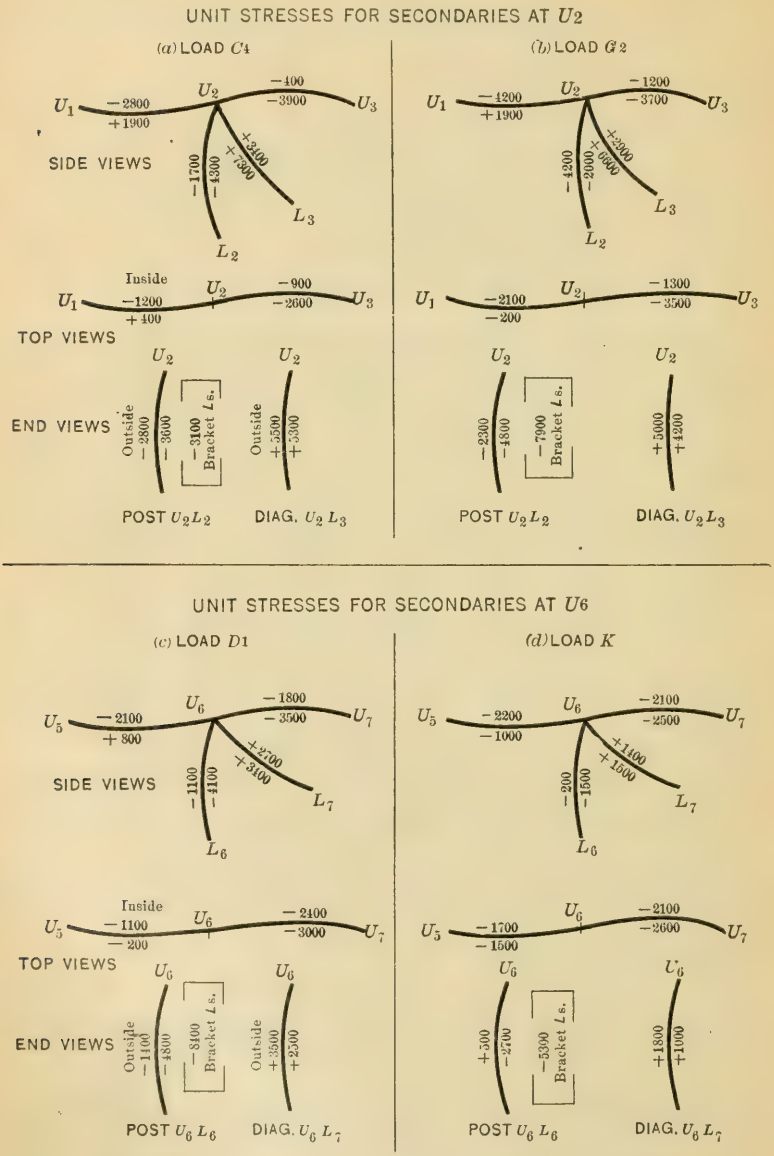


FIG. 6.—DIAGRAMS OF SECONDARY STRESSES, NIAGARA ARCH.

To insure that the railway speeds over the bridge shall not exceed the maximum of 17 miles per hour, a Bristol speed recorder, similar to that installed for the Pennsylvania Tunnels at New York City, has been provided.

Secondary Stress.

The general inspection of the Niagara Arch indicated that there were comparatively large secondary stresses. The smallest increment of stress of this kind was due to the weights of the members themselves, and another comparatively small amount was caused by the main hinge rollers being locked against proper action. The largest increments were from the distortion of the arch span under moving trains and partial live loads, from the curved sway brackets between the upper and lower floor-beams connecting to the main posts, and from the curved knee-braces connecting the girders and posts of the approach viaducts.

Strain-gauge measurements were made for secondary stresses, with 8-in. Berry instruments, and on points placed near the main gussets and also at other points where judgment indicated maximum values. The character and extent of the secondary stresses in the main posts and curved brackets of the arch span, are shown in Fig. 6 (*a*) to (*d*). One of the greatest variations in unit stress is shown in Fig. 6 (*b*), where it was 2 300 lb. on the outside of Post $U_2\ L_2$, 4 800 lb. on the inside of the post, and 7 900 lb. on the flange of the curved bracket.

The fact will also be noted from these diagrams that all the top chords have secondary stresses in both vertical and horizontal planes. Secondary stress in the horizontal planes was always of the same character, and on analysis this was found by the writer to be due to the force acting at the extreme end of the heavy main gussets, and induced by the bending from the curved brackets. The great secondary stresses induced by these curved brackets, however, was only one of the reasons calling for their reinforcement.

The secondary stresses in the posts of the 115-ft. spans and the viaduct posts were provided for by the reinforcing angles which were added to these members to increase the sectional area for the direct compression stress due to the new live load capacity of the structure. These angles were required for giving as large a radius of gyration as possible to the members; but, as providing for the secondary stresses, the angles required as short outstanding legs as possible in order

to keep down the fiber stress. The method of adding these angles is described under the head of practical strain-gauge applications.

The secondary stresses were not calculated for the main arch members, but were compared with the plotted curves of the secondary stresses as calculated for the Hell Gate Arch, a similar type of structure. Diagrams, four in number, similar to Fig. 7, were prepared for the top chords, the bottom chords, the vertical posts, and the diagonals. On each of these four sheets, the secondary stresses read by the strain gauges upon the Niagara Arch were plotted, and a curve was established similar to the combined dead and live load curve for the Hell Gate Arch. The panel lengths of the Niagara Arch were laid off to the same scale as that used for the Hell Gate Arch, at the bottom of the diagram, thus making it possible to compare directly the percentage of secondary stress for each member in each panel of the Niagara Arch. The probable error in any case is indicated as being on the safe side, and for all practical purposes the resulting secondary stresses as entered in the stress sheet may be considered within the possible error of the Niagara strain-gauge readings, or within 4 to 6 per cent. One feature must be carefully considered in applying secondary stresses such as these, that is, that they usually reach a maximum near the panel points, and are not necessarily co-existent with a maximum compressive stress due to the flexure of a long column.

The general results of the strain-gauge readings seem to indicate that the stress in built tension members is greatest toward the center and the stress in built compression members is least toward the center of the section, which emphasizes the method used on the compression members of the Hell Gate Arch to make them take stress at the center first, and distribute it to the outsides or flanges.

The fact is also demonstrated that there is a better distribution under heavy stress, than for light stresses, and this was emphasized in many of the readings on the Niagara Cantilever.

The distribution of stress in the eye-bar chords of the Niagara Cantilever was determined by reading with the Berry strain gauge and was found to be better under heavy total stress than with light total stress. Table 2 illustrates this for the light loads *A* and *B*, and the heavy load *G*, in the top chord $U_8\ U_9$, composed of fourteen eye-bars. The load *A* consisted of one 188-ton locomotive followed by twenty cars weighing 2 740 lb. per lin. ft., with only the train load

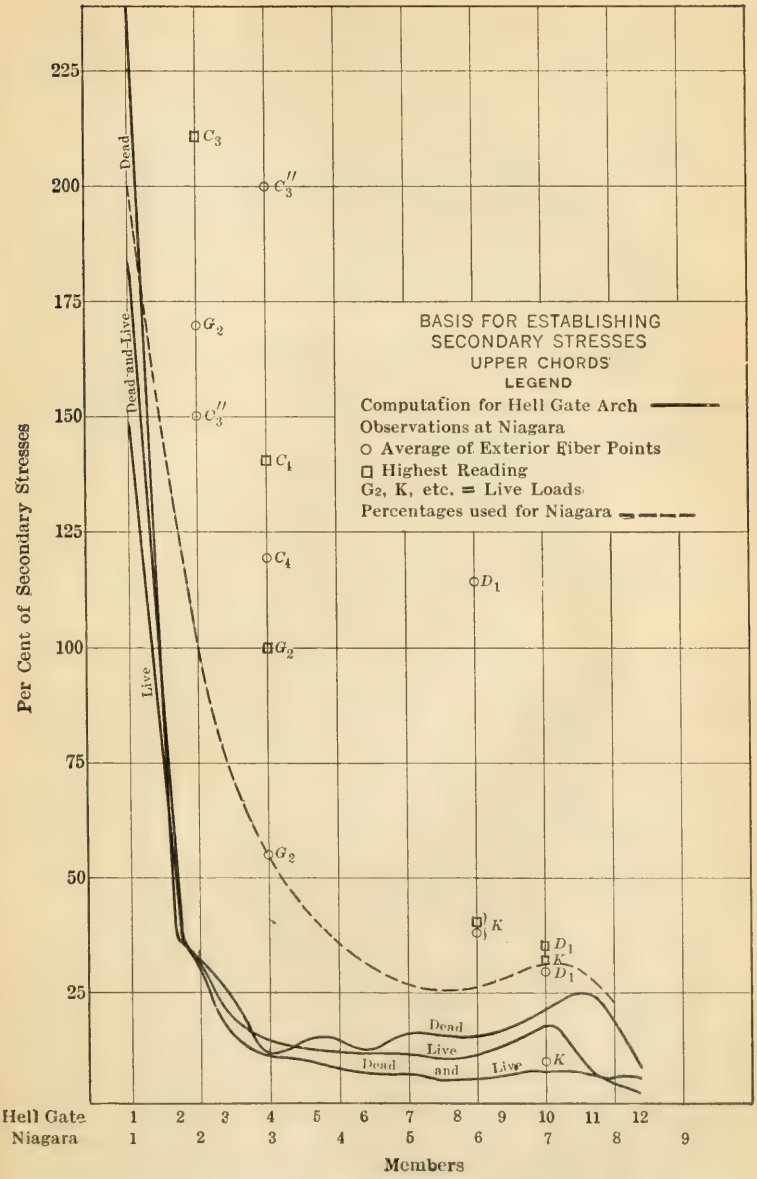


FIG. 7.—SECONDARY STRESSES IN NIAGARA ARCH AND HELL GATE ARCH COMPARED.

on the bridge; while Load *B* was the same train, with the locomotive on the suspended span and the train covering the cantilever and anchor arms. Load *G* was a 167-ton locomotive and six loaded cars weighing practically the same as for Loads *A* and *B*, but with the locomotive on the suspended span and the train load on the cantilever arm and suspended span, so located as to give maximum stresses in the bars of U_8 U_9, which was the tower panel of the top chord. The unit stresses for Loads *A* and *B* ranged from 300 to 1 300 lb. per sq. in., with an average of 700. The unit stresses for Load *G* ranged only from 2 300 to 3 300 lb. per sq. in., with an average of 2 800.

TABLE 2.—Niagara Cantilever Bridge, North Truss.

Unit Stresses in Top Chord U_8 U_9, Read with Berry Strain Gauge.

Load.	5 Bars 8 by 1¾ = 700 Sq. In.					4–8 by 1½ = 480 Sq. In.				5 Bars 8 by 1¾ = 700 Sq. In.					Average.
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
A	4 1 000	3 800	4 1 000	2 500	5 1 300	3 800	3 800	2 500	3 800	1 300	1 300	2 500	2 500	2 500	700
B	5 1 300	5 1 300	5 1 300	2 500	5 1 300	4 1 000	4 1 000	3 800	3 800	1 300	2 500	2 500	2 500	2 500	800
G	12 3 100	12 3 100	11 2 800	11 2 800	13 3 300	10 2 600	12 3 100	10 2 600	12 3 100	10 2 600	11 2 800	10 2 600	9 2 300	11 2 800	2 800

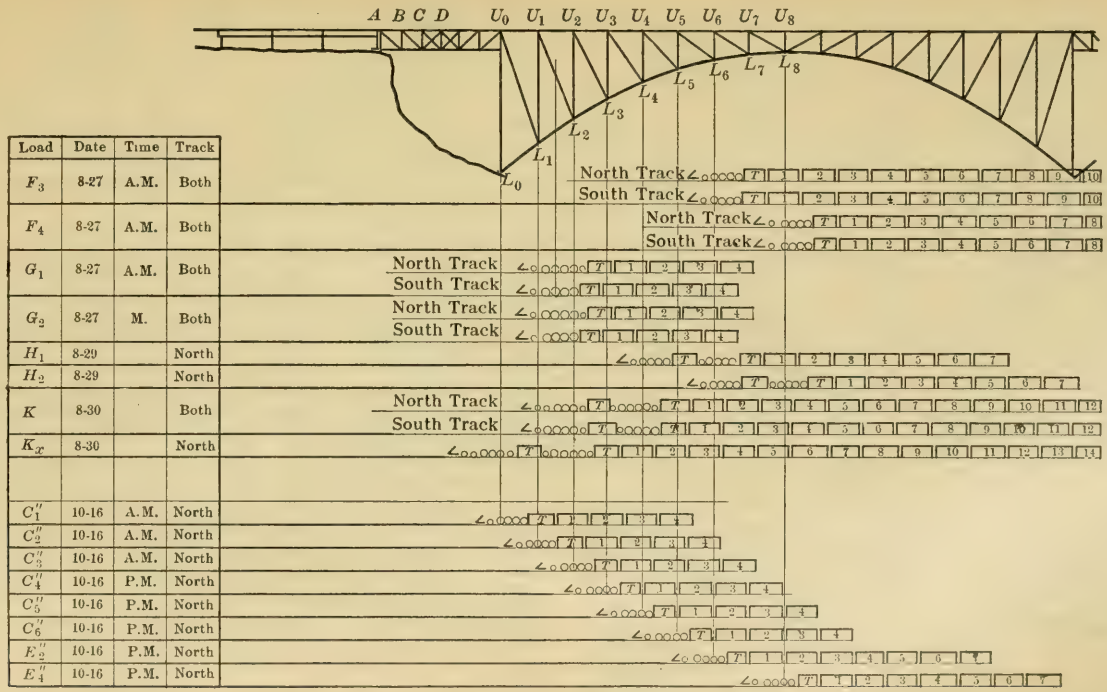
A careful study of the distribution of stress over the cross-section of either compression or tension members shows how little the supposed perfection of designs is realized in practice, both as to correctness of stresses and distribution. The stress measurements made at the same time on another structure, where the stringers rested on top of the floor-beams and were supposed to slide, disclosed the fact that one angle of a floor-beam flange was undergoing compression, while the other was in tension. The results obtained on the Niagara Arch were utilized only to the extent necessary for the re-designing of the structure, but it seems to be certain that comprised within the readings made there are the germs for many rectifications of the methods of calculation and design of large structures, and it is to be hoped that the results of this work will form the basis for future extensive readings on this arch.

Test Loads.

The test loads used by Mr. Roebling on the Suspension Bridge in 1855 have already been described, and from this 326-ton total load to the 1 670-ton load used by Mr. L. L. Buck on the arch span a great advance was marked. The 90-ton locomotives and 52.5-ton cars used to make up the test load, however, indicated that the assumed live load used in designing the new arch bridge was not taken heavy enough, and nothing less than Cooper's E-40 should have been adopted as it was presumably being used by some roads at about this time.

When trains were assembled for the 1918 strain-gauge readings, no difficulty was found in getting locomotives and cars nearly as heavy as E-50 loading, and no particular trouble would have been experienced in assembling trains fully equal to or heavier than E-50. The test trains, used over a period of nearly two months, were twenty-five in number and varied in average weight from 4 250 to 4 880 lb. per lin. ft. The weight of the cars alone varied from 3 840 to 4 320 lb. per lin. ft. Many cars passing over the bridge indicated also that a train load of 5 000 lb. per lin. ft. could easily be assembled, and would be considerably exceeded by actual trains before many years.

The test trains were located on the bridge during the strain-gauge readings, as indicated in the typical diagram, Fig. 8. The locations corresponded closely to positions for maximum stress in various members, as determined from a study of the influence diagrams. The length of the trains was also determined for giving maximum stresses for various members, in the locations fixed for the trains. The shorter trains consisted of one locomotive and four cars; the second of one locomotive and seven cars; the third of two locomotives and seven cars; the fourth of one locomotive and sixteen cars; and the longest of two locomotives with fourteen cars. The locomotives ranged in weight from 177 tons and 61 ft. 5 in. long, to 220.9 tons and 72 ft. 3 in. long. The heaviest locomotive likely to cross the arch is shown in Fig. 9. The cars were mainly steel hopper-bottom gondolas, 33 ft. 6 in. long, weighing empty about 18 tons, and loaded about 70 tons. There were a few larger cars, 42 ft. 6 in. long, weighing empty about 26 tons, and loaded about 100 tons. The trains used during the entire two months, however, were of an extremely uniform character, thus tending to great uniformity in the strain-gauge readings and increased accuracy. Practically no change was found in the strain-gauge read-



Load	Date	Time	Track
F_3	8-27	A.M.	Both
F_4	8-27	A.M.	Both
G_1	8-27	A.M.	Both
G_2	8-27	M.	Both
H_1	8-29		North
H_2	8-29		North
K	8-30		Both
K_x	8-30		North
C_1''	10-16	A.M.	North
C_2''	10-16	A.M.	North
C_3''	10-16	A.M.	North
C_4''	10-16	P.M.	North
C_5''	10-16	P.M.	North
C_6''	10-16	P.M.	North
E_2''	10-16	P.M.	North
E_4''	10-16	P.M.	North

LOCATION OF TRAINS FOR STRAIN GAGE READINGS.

FIG. 8.

ings on the arch members whether the first, second, or third driver was located at a panel point.

Strain Gauges.

The greater part of the work of measuring stresses was done by the use of the well-known Berry strain gauge, using the 8-in. gauge for secondary stress work, and the 20-in. gauge for primary stresses. These instruments (Fig. 10) have the side strips made of "Invar" metal, the coefficient of expansion of which is only about one-twenty-eighth that of carbon steel, and the handling during use does not seriously affect the accuracy of the readings. One end has a fixed conical steel point, and the other end a movable conical steel point attached to a bell-crank lever. The long arm of this is attached to the spindle of an Ames gauge dial, and the lever has a multiplication of five times, so that while readings are made directly to 0.0002 in., they may be estimated to 0.00002 in.

All the instruments were carefully calibrated to determine their error, by the use of a standard micrometer screw, and all three Berry instruments were found to read slightly high, one 3%, one 3½%, and the greatest error was 4% on the third gauge. These errors were of course constant, and as they were far within the degree of accuracy desired, no correction for this has been made in the results as given in the tables.

The Howard gauge (Fig. 10) used on the Hell Gate Bridge was kindly

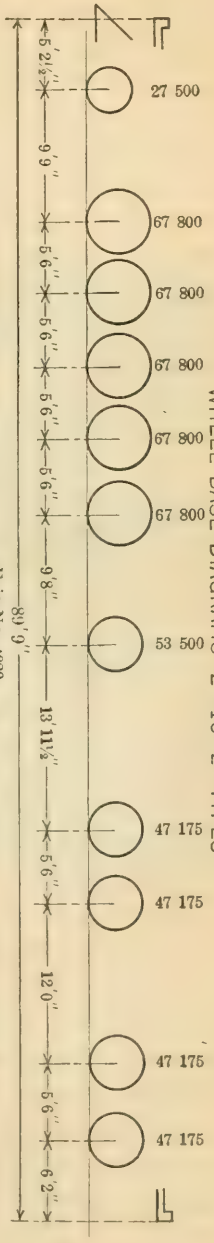


Fig. 9.—Heaviest Erie Railway Locomotive Allowable Over Niagara Arch.

loaned by Gustav Lindenthal, M. Am. Soc. C. E., for the purpose of making some comparative readings to determine the respective accuracy of the Berry, the Howard, and the Fuller-West instrument described in the following paragraph. The Howard gauge has one fixed and one movable conical steel point, the movable one having a horizontal sliding movement and being attached to a rod sliding in the tube which carries the fixed point. The distance between the two points is read by a micrometer contact screw at the end of the tube next to the movable point, and reads to 0.0001 in. There are two milled heads with a spring ratchet between them, which slips where contact is made. The inside milled head may be used, the operator depending on the sense of touch for the contact, or the ratchet outside head may be used. This instrument, not being of Invar metal is covered with leather for protection from heat in handling.

The Howard gauge obviously cannot be used except for static load readings, and while the Berry instrument was used with moving loads and for impact readings, the excessive vibration of the needle of the Ames dial made it difficult for the observer to get accurate readings of the maximum stress at all times, and, therefore, many check readings were required. This led A. H. Fuller, M. Am. Soc. C. E., to an investigation of the "Last Word" dial, made in Cleveland, Ohio, and to the construction of a much more reliable instrument, using this dial, by Mechanician West, of Lafayette College, embodying Professor Fuller's ideas and some suggestions from the writer. This instrument, called the "Fuller-West", is shown in Fig. 10 (the two lower figures), and is provided with fibroid handles to obviate trouble from change of temperature in handling. The joint of the movable point lever is a double cone pivot and much more accurate readings of moving load stresses can be made, and for static stresses it has been found to be more accurate than the Berry, or, in other words, where from eight to ten readings were made on a point with the Berry, a less number and one check reading with the Fuller-West instrument gave just as reliable results.

The accessory instruments are practically the same in all cases, so only those used in the Niagara Arch readings and for the stress readings made by the writer and his assistants on the Michigan Central Niagara Cantilever, will be briefly described, those for the Howard instrument having been fully described in the paper by D. B. Stein-

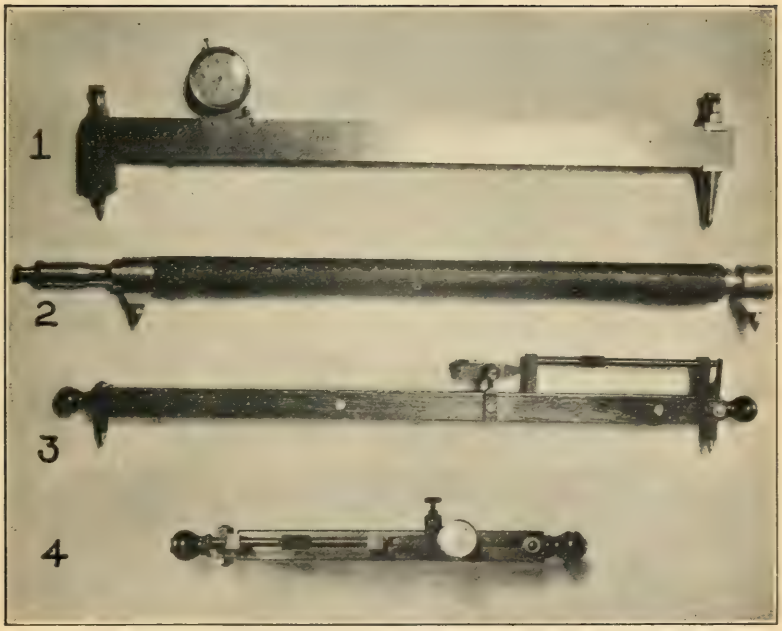


FIG. 10.—STRAIN GAUGES.

1.—BERRY 20-IN. GAUGE.
2.—HOWARD 20-IN. GAUGE.
3.—FULLER-WEST 20-IN. GAUGE.
4.—FULLER-WEST 8-IN. GAUGE, TOP VIEW.

man, M. Am. Soc. C. E., entitled "Stress Measurements on the Hell Gate Arch Bridge".*

Standard Bars.—Two bars were made from pieces of ¾-in. gas pipe, long enough to allow caps to be screwed on to the ends and not interfere with the gauge points 20 in. apart. The caps were screwed on tight and were then ground down flat on one side to give a flat base to rest upon. In the center of each a slot was cut on the opposite side and a thermometer placed inside where it could easily be read. These standard bars were marked *A* and *B*.

In making observations a reading with the gauge was made on the standard bar, a group of points on members read, and then a check reading was made on the standard bar. The thermometer was read and recorded each time the standard bar was read.

Corrections for Temperature.—These were made from readings of special Fahrenheit thermometers which had been calibrated by comparison with a standard laboratory thermometer. The thermometers, twelve in number, were fastened in close contact to members of the bridge and the bulbs bedded in putty to protect them from the air. The locations were changed as the work progressed. It was not considered necessary to have a thermometer on each bar or member when it was being read for stress, but the temperatures were read at the same hour for members similarly located. Very large differences, up to 27°, were found on opposite sides of a member—in the sun or away from it. The corrections for temperature were made by taking the coefficient of expansion of the steel in the bridge as 0.0000065. All the thermometers were kept in the shade.

Members Read for Stress.

The members read for stress are shown by the heavy lines in Fig. 11. The members of the northeast quarter of the arch, down stream on the American end, were read most extensively, as that was more shaded from the sun. Members in the other quarters of the arch were read in sufficient number to give a check on this northeast quarter.

Gauge Points.—These were located on the upper part of the arch so as to be easily reached from the highway deck, and by means of low staging or ladders. The points for secondary stress were located near the main top gussets and in large enough numbers to obtain results

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1040.

for both longitudinal and transverse secondary stresses. Points were located on Post U_2 L_2 at two elevations below the highway deck, and, for the arched chord, they were located inside the chord members. The location of the temporary and permanent gauge points was recorded in tabular form.

Spots about 1½ in. in diameter were cleaned of paint and rust and placed either 8 in. or 20 in. apart as needed. The holes for the "points" were drilled with a Yankee drill about $\frac{1}{10}$ in. deep, and with a No. 56 drill. The burr was removed by a flat-angle countersink and finished by rubbing with a center punch. One hole of each pair was finished before the other was marked, by using a bar with center punches spaced 8 in. apart. The holes were protected when not in use by filling them with vaseline.

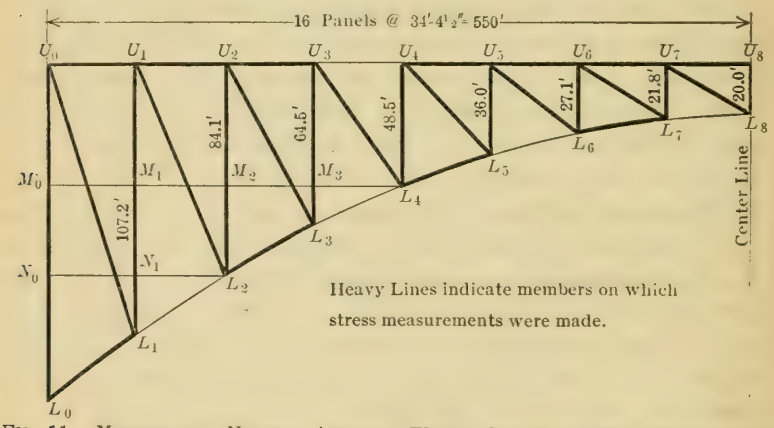


FIG. 11.—MEMBERS OF NIAGARA ARCH TO WHICH STRAIN GAUGES WERE APPLIED.

Permanent Gauge Points.—These were placed on members, as indicated in a list in the Company files, so that readings on such members could be checked up for different loadings at any future time. These points were set in the bottom of ½-in. holes ¼ in. deep, which were tapped and provided with cap-screws having a lead washer under the heads. The holes were filled with vaseline before the cap-screws were put in place.

Modulus of Elasticity.—The modulus of the steel in the bridge is the basis on which the strain-gauge readings have been reduced, and this is undoubtedly 29 000 000 lb. per sq. in. The character of the material was specified to be steel having a strength of from 60 000 to

68 000 lb. per sq. in. and an elastic limit of at least 33 000 lb. per sq. in. It should be noted, however, that the steel was really only of 30 000 lb. elastic limit, as the value determined by the drop of the beam of the testing machine was what is now termed the yield point, and 30 000 lb. has been used as the real value of the elastic limit.

The original tests of the material were obtained by the writer from R. W. Hunt and Company, of Pittsburgh, Pa., successors to G. W. G. Ferris and Company, the original inspectors of the steel in 1896 and 1897. These tests show the yield point to be as specified for the elastic limit, and, therefore, the elastic limit used in determining the proper load to put on the arch and approaches, has been taken at 30 000 lb.

The stress measurements or readings were recorded after the manner shown on the typical sheet, Table 3. In the first column, headed "Load", *D L* represents the static train load, and the letters refer to the standard

TABLE 3.—Typical Strain-Gauge Record.

Dead and Live Load Averages, in Dial Units.

Load.	Date.	Time.	Temperature.			Standard.	Points.					
			Standard.	Arch.	Member.		1	2	3	4	5	6
Vertical U_0 L_0, Bottom.—Gauge Length 20 In.												
D.L.	10-14	11.00	49	49	49	B	— 41	— 22	— 35	— 42	.....	.....
							— 41	— 22	— 35	— 42	.....	.....
C_1''	10-16	8.40	53	53	53	B	— 26	— 8	— 22	— 29	.....	.....
							— 26	— 8	— 22	— 29	.....	.....
Vertical U_0 L_0, Top.—Gauge Length 20 In.												
D.L.	8-15	11.30	84	85	83	A	— 56	— 43	— 49	— 39	— 66	+ 18
							— 57	— 43	— 49	— 39	— 66	+ 18
D.L.	8-20	7.40	61	61	60	A	— 52	— 41	— 46	— 37	— 64	+ 22
							— 53	— 42	— 47	— 38	— 65	+ 21
Dead load mean				70	.....	.....	— 55	— 42	— 48	— 38	— 66	+ 20
A_1	8-15	9.20	65	66	65	A	— 42	— 27	— 40	— 30	— 53	+ 26
							— 42	— 27	— 40	— 30	— 53	+ 26
C_1	8-20	8.35	64	63	62	A	— 35	— 22	— 36	— 28	— 51	+ 29
							— 36	— 23	— 37	— 29	— 52	+ 28
C_1'	8-20	9.00	65	64	63	A	— 34	— 22	— 36	— 28	— 50	+ 29
							— 35	— 23	— 37	— 29	— 51	+ 28
Vertical U_1 L_1.—Gauge Length 20 In.												
D.L.	10-13	11.50	60	62	62	B	— 79	— 70	— 63	+ 96	+ 24	— 55
							— 78	— 69	— 62	+ 97	+ 25	— 54
C_2''	10-16	9.10	52	53	53	B	— 64	— 54	— 58	+102	+ 34	— 42
							— 64	— 54	— 58	+102	+ 34	— 42

trains as indicated on Fig. 8, the details of each train being recorded in a record of "Test Loads". The date of the readings is given in the second column, and the exact hour in the third column. This enabled exact co-ordination of temperatures taken from sheets similar to Fig. 12. The first of the three columns headed "Temperature" gives the reading of the thermometers in the standard bars. Two standard bars were needed, one for each observation crew, and these were standardized one with the other, so that readings could be compared, and with duplicate bars kept in the fireproof vault, so that in case of loss they could be replaced. The second temperature column headed "Arch" contains the temperature as reduced from a large number of thermometers at various places on the structure, and represents the values used in making corrections for temperature stresses. The third temperature column gives that of the member read, and was

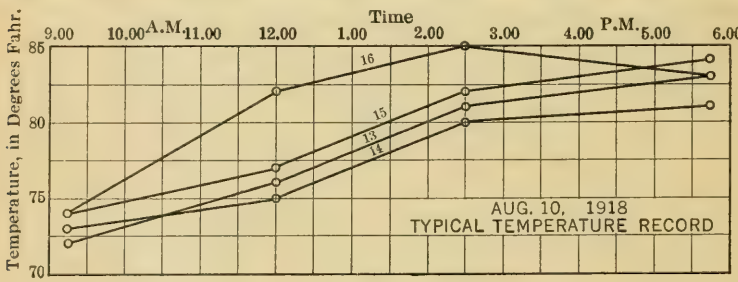


FIG. 12.

determined from thermometers fastened directly to the member or to one adjacent.

The results in the numbered columns are the strain-gauge dial units for the corresponding numbered point on the cross-section of the member in Table 4. The figures given are the averages of all the readings for that point, which were always two or more independent readings. Two temperature corrections were made for some of them, one, where there was a difference between the temperature of the member and the standard bar, and the other due to the temperature stresses in the arch. The large figures are the corrected ones, and the small ones the originals, thus making it possible to check the temperature corrections at any time.

The tables of live load stress measurements, Table 4 being typical, give the deformations in dial units which are direct deductions from

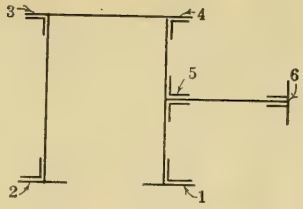
the dead and live load averages, and the stresses in pounds per square inch which have been computed from the deformations. An average of all the unit stresses in one member has been made and the area is given, thus making it possible readily to determine the total stress in the member.

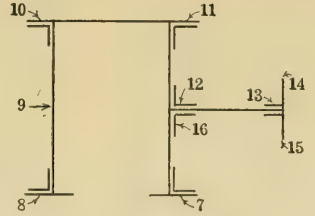
About 400 gauge points were read on the members shown in heavy lines in Fig. 11, and upward of 5 000 individual and check readings made to establish the unit stresses. The readings were made in the cooler hours of each day so far as this was possible, but careful records of the temperatures were made as noted, and kept on about fifty diagrams similar to Fig. 12.

The comparisons made by calculating the stresses for certain members due to the test train used in the location for maximum stress, indicated that the results obtained by the strain-gauge readings were in error only from 4 to 6 per cent. This, of course, would not be true for members having very low unit stresses, as the unit stress must be of considerable amount to obtain an accurate result with the strain gauge; or, in other words, the error in the strain gauge is largely fixed in amount, and would, therefore, increase as a percentage the smaller the unit stress.

The practical uses for the strain gauge in actual construction or maintenance work on bridges were found to be many. Pending the substitution of stiff sway bracing instead of the rods in the 550-ft. arch span, it was necessary carefully to adjust all of them, some 96 in number, previous to the fall and winter of 1918; the former most careful adjustments by careful judgment were found to have put very little initial tension into some of the rods and too much into others. Some were found with 27 000 to 28 000 lb. per sq. in., or practically up to the elastic limit of the bars, which were of iron. They were all adjusted with the use of the strain gauge for an initial stress of 7 000 lb. per sq. in.

The movement of the segmental hinge rollers of the main arch at the skewbacks was so slight as not to be discernible by the eye, but by establishing gauge points, one on the upper steel casting and one on the lower, it was possible to detect movements with the strain gauge, due to moving trains, of from $\frac{1}{256}$ to $\frac{1}{100}$ in., and readings at various temperatures established the total movement for extremes of temperature and loadings at practically $\frac{1}{10}$ in. This is quite small and extends over an





Measurements for Live Load Secondaries.

Gauge Length, 8 In.

Member.	Load.	Points. 7	8	9	10	11	12	13	14	15	16	Average.
$U_2\ L_2$	C_4	— 6.9 —5 000	— 4.9 — 3 550	— 4.4 — 3 190	— 2.3 — 1 670	— 2.5 — 1 810	— 4.6 — 3 330	— 5.1 — 3 690	— 3.5 — 2 540	— 4.0 — 2 900	— 6.0 — 4 350	— 3 200
	G_2	— 7.2 —5 220	— 4.4 — 3 190	— 3.7 — 2 682	— 1.3 — 990	— 4.3 — 3 120	— 6.6 — 4 780	— 13.1 — 9 500	— 9.9 — 7 180	— 9.8 — 7 110	— 8.5 — 6 160	— 5 000
	C_3''	— 4.3 —3 120	— 2.5 — 1 810	— 5.2 — 3 770	— 1.8 — 1 300	— 2.2 — 1 590	— 3.8 — 2 750	— 6.9 — 5 000	— 4.3 — 3 120	— 4.4 — 3 190	— 5.4 — 3 910	— 2 960
$U_6\ L_6$	D_1	— 8.5 —6 160	— 3.0 — 2 170	— 3.5 — 2 540	+ 0.6 + 430	— 3.7 — 2 680	— 5.8 — 4 200	— 16.0 —11 600	— 10.1 — 7 320	— 9.5 — 6 890	— 8.5 — 6 160	— 4 930
	K	— 4.4 — 3 190	+ 0.4 + 290	— 0.6 — 430	+ 2.0 + 1 450	— 2.5 — 1 810	— 3.1 — 2 240	— 11.0 — 7 980	— 6.9 — 5 000	— 6.8 — 4 930	— 4.9 — 3 550	— 2 740

TABLE 4.—Unit Stresses in Arch Posts from Strain-Gauge Readings.

Stresses, in pounds per square inch; — sign = compression.

Deformations in Dial Units—1 Dial Unit = 0.0002 In.

Live Load Stress Measurements.

Gauge Length, 20 In.

Member.	Load.	Points. 1	2	3	4	5	6	Average.	Area, in square inches.
Top $U_2 L_2$	A_3	— 19 — 5 510	— 17 — 4 930	— 10 — 2 900	— 12 — 3 480	— 16 — 4 640	— 13 — 3 770	— 4 205	91.11
	C_3	— 21 — 6 090	— 20 — 5 800	— 10 — 2 900	— 12 — 3 480	— 15 — 4 350	— 15 — 4 350	— 4 495	
	K	— 17 — 4 930	— 15 — 4 350	— 9 — 2 610	— 12 — 3 480	— 15 — 4 350	— 14 — 4 060	— 3 969	
$U_2 L_2$ Below F	C_3''	— 15 — 4 350	— 16 — 4 640	— 13 — 3 770	— 13 — 3 770	.....	.....	— 4 130	91.11
$U_2 L_2$ at M_2	C_3''	— 14 — 4 060	— 18 — 5 220	— 17 — 4 930	— 13 — 3 770	.....	.....	— 4 495	91.11
$U_1 L_1$ $S E$	C_5	— 20 — 5 800	— 17 — 4 930	— 7 — 2 030	— 9 — 2 610	— 15 — 4 350	— 19 — 5 510	— 4 205	78.71
	L_2	— 8 — 2 320	— 5 — 1 450	— 2 — 580	— 8 — 2 320	— 9 — 2 610	— 8 — 2 320	— 1 930	
$U_6 L_6$	D_2	— 15 — 4 350	— 19 — 5 510	— 8 — 2 320	— 5 — 1 450	— 8 — 2 320	— 5 — 1 450	— 2 900	74.71
$U_7 L_7$	D_2	— 18 — 5 220	— 15 — 4 350	— 7 — 2 030	— 8 — 2 320	— 14 — 4 060	— 14 — 4 060	— 3 670	75.21

entire year, so that no fear is now entertained as to the rollers creeping, and although this has been corroborated by careful inspection extending over a year and a half, such rollers should have gearing similar to that used on the Ohio River Beaver Bridge. The addition of such gearing was first contemplated at Niagara, but hook indicators were attached to the side plates, which have a clearance of $\frac{1}{8}$ in. from the steel castings at 50° Fahr., and so if any creeping occurs it can be readily detected.

The reading of the dead load stress in the center top chord, $U_7\ U_8$, of the arch span was accomplished in 1918 by the strain gauge, and by using a heavy toggle as described subsequently; and as this toggle was improved and duplicated for changing the center shims in 1919, there was available the toggle slabs and screws for other adjustments necessary in the reinforcement of the members. The method of reinforcing the intermediate and shore end posts of the 115-ft. approach spans, contemplated the placing of expensive falsework down to the rock banks of the Gorge, to relieve the posts of dead load while the reinforcing angles were being riveted to the old members. With an **A**-toggle (Figs. 13 and 14), abutting on the reinforced shelf at the bottom of each post and on steel blocks to distribute the pressure over the flange of the top chord, the dead load on the posts was relieved by screwing up the toggle until strain-gauge readings showed the posts to be free. This required from four to five dial units on the strain gauge. The toggle, when trains crossed, carried also a portion of the live load from the opposite track, as only two toggles were used at a time and placed on the posts of the trusses adjacent to the abandoned track. The fitting up bolts had to be loosened before the toggles were tightened, and drawn up again just before riveting began, to give proper adjustment.

The eight counters in the 115-ft. spans were replaced with **U**-counters having double the section, and the new rods were set with the strain gauge to have an initial tension of about 3 000 lb. per sq. in.

The greatest value of the strain gauge in the practical work on the arch revision was, of course, the checking up of the actual dead load stresses, which, on account of the introduction of the adjusting shims, did not agree with the theoretical stresses; an accurate checking of the distribution of stress over the cross-section of built tension and compression members; the checking of secondary stress; and the effect of impact at varying speeds.

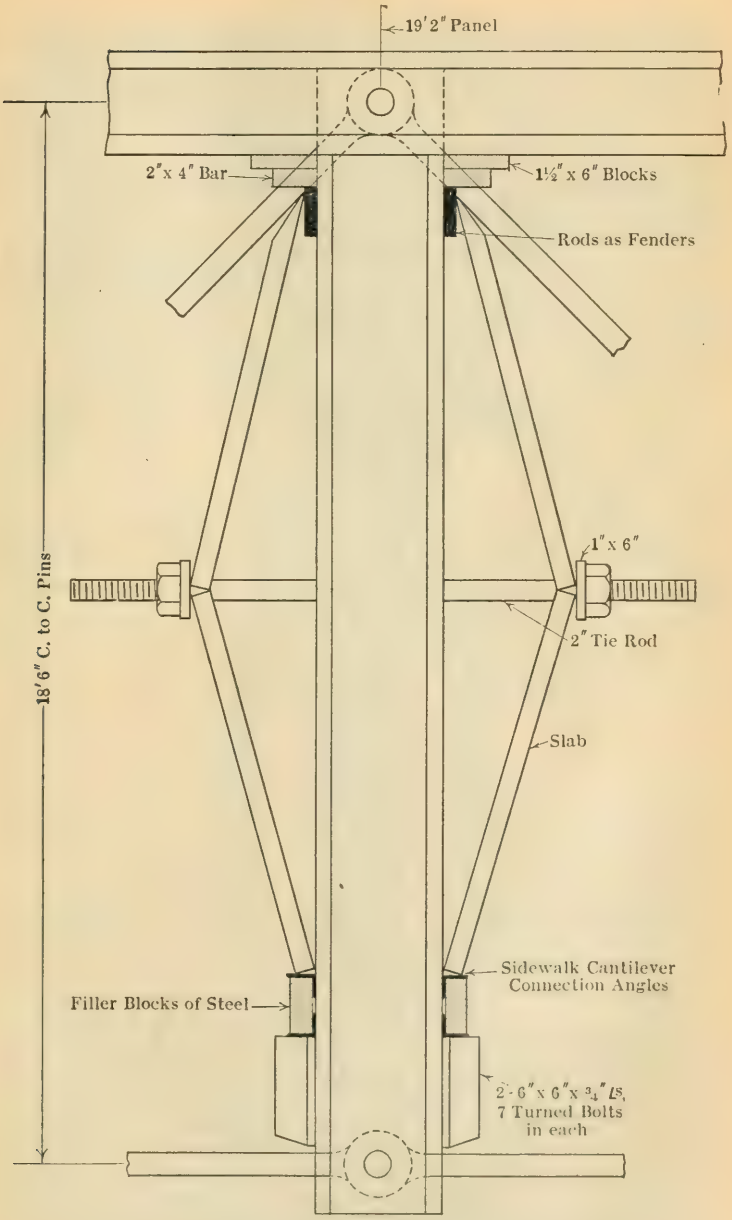


FIG. 13.—TOGGLE APPLIED TO POST OF 115-FT. TRUSS.

Unit Stresses.

The fact has been recognized for some time that for heavy, special, and long-span bridges, the Profession is no longer warranted in using the low values for unit stresses that are prevalent for less important shorter spans in which the secondary stress and closely accurate impact stresses have not been calculated or determined. Some authorities believe that where all stresses have been taken into account, unit stresses closely approaching the elastic limit should be used, with a loading that is quite sure not to be exceeded during the life of the structure, such as Cooper's E-90 loading or one similar to the composite loading proposed in this paper.

The fact remains, however, that there are still some uncertainties as to the uniformity of the elastic limit and the ultimate strength of materials, be they steel or steel alloys, as to the exact amount to be allowed for impact and vibration and as to the distribution of stress over the cross-section of the members, or stress transmission by means of pin and riveted joints. Therefore, for the Niagara Arch revision, after determining the minimum value of the elastic limit of the steel as shown by the somewhat extensive original tests thereof, it was decided to adopt unit stresses that are practically two-thirds of the elastic limit of 30 000 lb. per sq. in., as follows:

Tension

Built truss members	20 000 lb. per sq. in.
Eye-bar members	22 000 " " " "
Girder flanges, net	18 000 " " " "
Laterals of shapes	26 000 " " " "

Shearing

Girder webs	15 000 " " " "
Rivets and pins	15 000 " " " "

Bearing

Rivets and pins	30 000 " " " "

Compression

Girder flanges	$20\,000 - 200\,\frac{l}{b}$

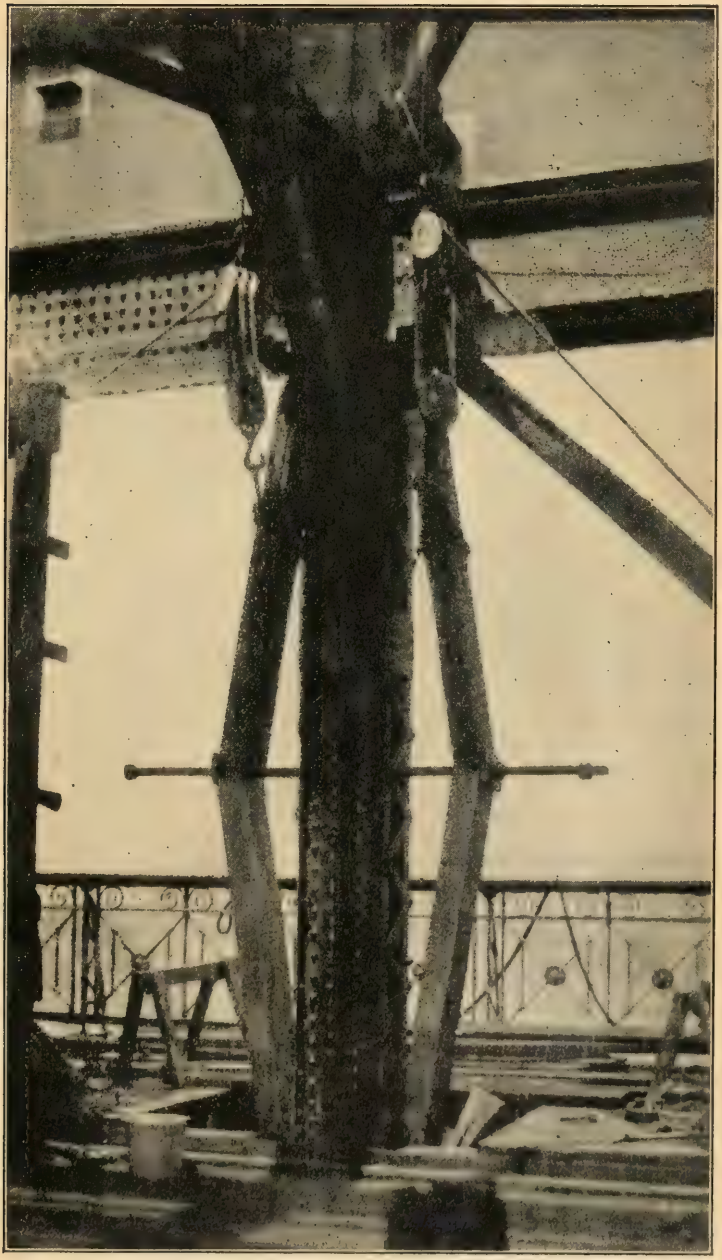
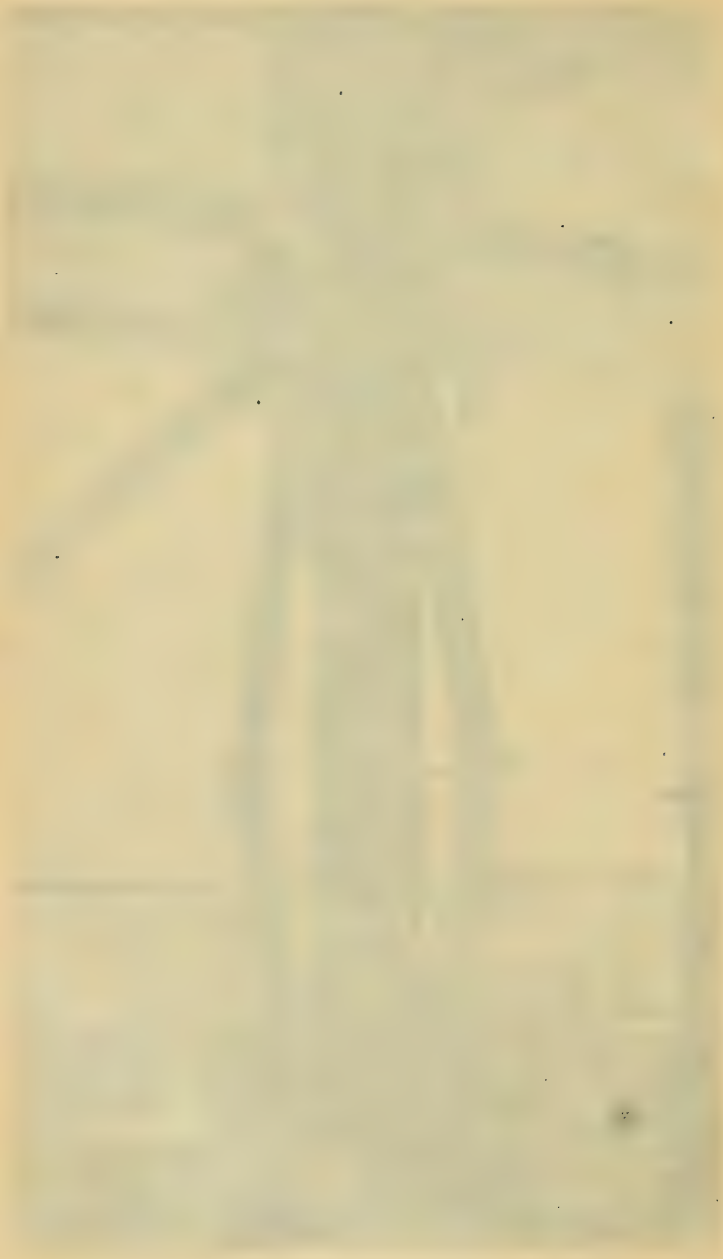


FIG. 14.—TOGGLE FOR RELIEVING DEAD LOAD STRESS IN POSTS OF 115-FT. SPANS.



Compression units for built chords and posts, were taken from diagrams similar to Fig. 15, using in each case the curves for an elastic limit of 30 000 lb.; while curves for nickel steel were established for use in future extensions of the bridge, where such material might be adopted. All these curves were established by plotting the values for column unit stresses as given by Mr. Lindenthal, in the specifications for the Hell Gate Arch, and then drawing the smooth curve which closely averaged them. The formula:

$$p = 23\,000 - 100\,\frac{l}{r}$$

represents closely one of these curves as shown in Fig. 15 and at the top

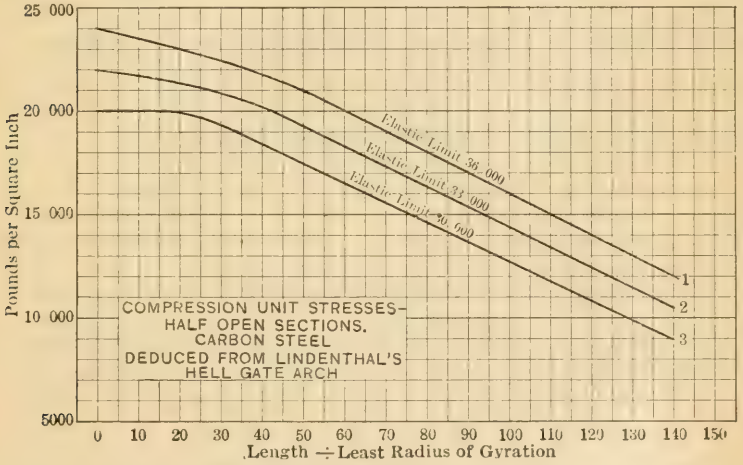


FIG. 15.

of Plate XXXVII, the maximum limit being fixed at 20 000 lb. per sq. in. and the maximum value of $\frac{l}{r}$ at 135.

The formula originally proposed for the Quebec Cantilever when all stresses, including secondaries and impact, were included, was:

$$p = 22\,000 - 88\,\frac{l}{r}$$

as shown in Plate XXXVII, which also shows the plotting of most of the principal column formulas, and which was made the basis of the adoption of the formula used for the Niagara work. The greatest number of these, in fact all those that are in general use, lie in a zone bounded

by $p = 12\,000 - 60\,\frac{l}{r}$ and $p = 18\,000 - 60\,\frac{l}{r}$, the average being the straight line formula, $p = 15\,000 - 60\,\frac{l}{r}$. The study by O. E. Hovey, M. Am. Soc. C. E., of the tests of columns made under the direction of the Special Committee on Steel Columns and Struts of the Society, shows that the ultimate strength of the light and heavy sections tested is represented by the respective formulas:

$$\text{Light columns, } p = 38\,000 - 70\,\frac{l}{r};$$

$$\text{Heavy columns, } p = 35\,000 - 72\,\frac{l}{r}.$$

These are plotted on Plate XXXVII for a safety factor of 2, and would indicate that all the present column formulas reduce unit stresses too fast for large values of $\frac{l}{r}$ and that for a two-thirds value of the average of the two Hovey formulas, we have practically:

$$p = 24\,000 - 47\,\frac{l}{r}$$

which gives much higher values proportionately than any of the ordinary formulas, especially for large values of $\frac{l}{r}$ or for very slender columns; and also indicates that the unit stresses used for columns in the Niagara revision were in comparison very conservative for the values of $\frac{l}{r}$ which existed.

The recent discussions before various technical societies and in the columns of the engineering journals indicate a strong preference for the retention of the straight line formula whenever a more logical value can thus be determined, and a careful study of the question shows that at least we should have a most careful review of all column tests made in the last thirty years, as the basis for a rational formula; and that it is preferable that more tests of columns of large size and careful design be made in a most painstaking and thorough manner. When such checked data are available, then unit stresses for columns can be tabulated once for all for safe end conditions, and no reason logically exists for caring what form the plotted line or curve may take, except that for convenience in memorizing and for use when tables are not at hand, some simple equation will still be desirable.

Preliminary Methods of Calculation for 1918 Revision.

The following methods were applied in making calculations for the 1918 revision of the Niagara Bridge.

Arch Span.

Dead Load.—The dead load shall be taken at the value for the center top chord that will best assist the lowering of the stresses in the top chord, posts, and diagonals, and at the same time not overload the arch chord to an unsatisfactory degree. This will presumably be a value between 200 000 and 300 000 lb. for U_7 U_8 (300 000 lb. was finally adopted).

Live Load.—The live load shall be Cooper's E-60 trains on both tracks, and on account of the peculiar terminal nature of the service on this structure, they shall both be considered headed the same way.

Impact.—The impact shall be taken at 50% of the live load stress impact as determined by the old American Railway Engineering Association formula on the above amount of live load on each truss, based on tests of 1918 (or for a speed of 17 miles per hour).

The stresses shall be computed for the maximum position with both tracks loaded, and 90% of this value shall be used for the total stresses in both trusses, one-half of this to be taken for the stress on the members of one truss.

Wind and Lateral Stresses.—The wind and lateral stresses shall be shown separately, but shall be considered as combined and shall not be used in determining the final unit stresses unless the combined amount of such stress on any member exceeds 25% of the total of all the other stresses on the member; and then only the amount in excess of such 25% shall be used in determining the unit stress.

Floor-Beams and Stringers.—The stresses in the floor-beams and stringers shall be determined by using the full value of the impact as determined by the old A. R. E. A. formula (for E-60), on account of the very severe service wear of this part of the structure and excessive corrosion (50% used for E-70).

115-Ft. Approach Truss Spans.

The live load on the trusses of these spans shall be taken as 100% on both tracks, but the impact shall be taken at 50% of the old A. R. E. A. formula, as for the arch span.

The wind and lateral stresses shall be considered, as provided for the arch span, except in the case of the reinforcing for the intermediate posts, in which case the whole of the bending stress may be taken account of, if so elected or considered necessary on final analysis.

The impact on the floor-beams and stringers shall be considered in the same manner as for the same members of the arch (50% used for E-70).

Approach Girders and Columns.

The impact on the approach girders and columns shall be considered in the same manner as for the floor-beams and stringers of the arch span and of the approach truss spans (50% used for E-70).

Reinforcement of Floor System.

The railway floor-beams on the arch rest on the top chords and are riveted to them; but additional rivets, seven in number, were added at each end. Large curved brackets for sway bracing are riveted to the beams and to the vertical posts, extending down to the bottom of the highway floor-beams which are riveted between the brackets. This entire construction was figured as a rectangular frame, and it was found that it was unnecessary to reinforce the flanges of the beams for the E-70 loading, but it was found necessary to add 10 by ½-in. plates to the flanges of the brackets. The plates on the inside of the brackets were extended along the bottom flanges of the beams to provide enough rivets to take up the stress carried by the plate, and attached to the beam flange by an I-section of plates and angles.

The values of the horizontal thrust, H, were determined by the Greene method, applying the formula originally used in figuring the arch stresses. This is a very sensitive formula and great care was used in calculating the value for each panel load, and all the calculations were thoroughly checked, so that it is certain that no sensible error was made in this basic matter. This was regarded as particularly important, due to the fact that much doubt was expressed by those discussing the paper describing Mr. Buck's methods as used in 1896, as to the correctness of the value of H originally used.

Arch Stress Readjustment.

The very extensive investigation made by the writer, in preparing a discussion* of the paper on "Economics of Steel Arch Bridges", indicated that for such a structure as the 550-ft. Niagara Arch it would have been economical to have designed it as a three-hinged arch for dead load, but also to act as a two-hinged arch under live load. The fact that Mr. L. L. Buck had added a section to the curved chord equal to one-half the difference between the section required in a two-hinged and that for three-hinged arch, showed the advisability of determining in some manner the actual dead load stresses existing in the arch ribs after the introduction of the 1-in. shim for closure of the arch when the erection of the span was completed. This operation was partly described in the paper published in 1897, but, briefly, it may be stated that the

* Page 47.

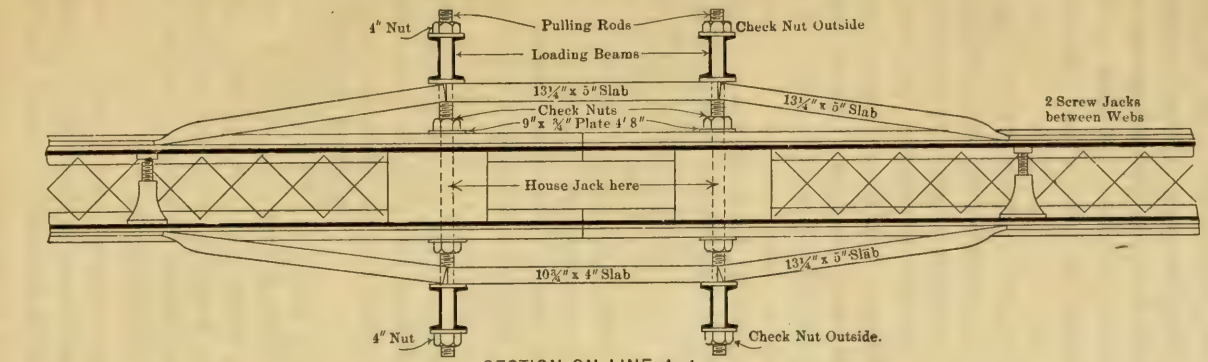
ribs did not close at the center top chord joints by about $\frac{1}{2}$ in. The arch was opened still further by the large shore toggles and a small adjusting toggle on each chord, so that a 1-in. shim was placed in each opening, which it was calculated would give the final 572 000-lb. stress in each chord as required for a true two-hinged arch. No way was provided at that time to measure the stress, and while it was at once evident that the small adjusting toggle would not have produced so much stress, it was later learned by the writer that the large shore toggle aided in producing a stress of 689 000 lb. in each chord, as measured on October 13th, 1918, for the north center chord, U_7 U_8, in the following manner:

The plates riveted on the center top chord webs for skewbacks in 1896 were reinforced for the load to be picked up by the new toggle by driving additional rivets in the front edge, and by bolting with 1-in. turned bolts, near the end a 4 by 1-in. bar at each point, to prevent the buckling up of the plate under pressure (Fig. 17). The adoption of a king-post toggle, such as was originally used, was out of the question owing to interference with the beams and bracing on the inside of the chords, and a heavy queen-post toggle was designed, as shown in Figs. 16 and 17, of sufficient strength to pick up a 400-ton load. This was supplemented by placing house-jacks inside the chords to counteract the transverse thrust at the skewback contact. The toggle, as modified slightly for actually changing the shim, is shown in Fig. 16.

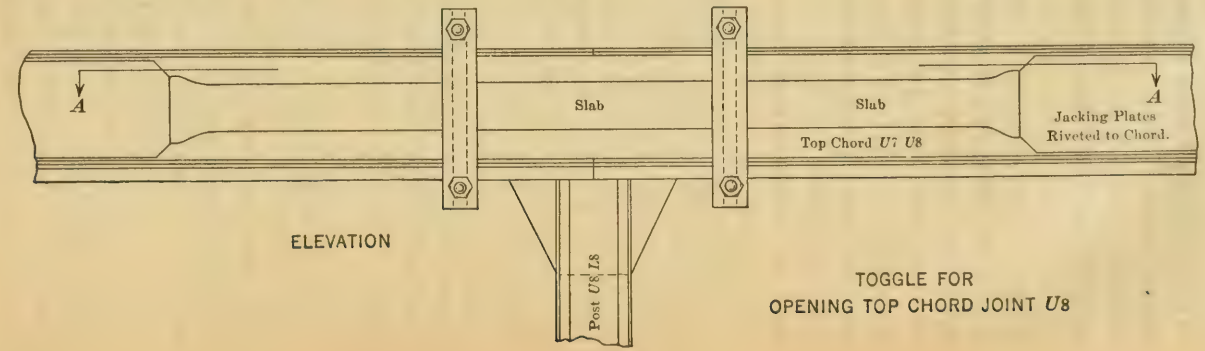
Gauge points were placed on the chords between the center joint and the skewback points, so that when the toggle was screwed up, after cutting out all splice rivets on one side of the joint, this portion of the chord was free, even from its dead load stress, and the difference between the strain-gauge readings before and after the joint was opened gave the actual unit stress. Traffic was suspended during the time of operating the toggle but a light passenger train was allowed to use the opposite track before the toggle had been entirely slacked away.

The use of the toggle in 1918 for stress measurement indicated, from the bending of the heaviest wrenches at hand, that screws with more threads per inch should be substituted the next year for actually changing the shims. The use of wood blocking for keeping the toggle in line at the "hips" proved very unsatisfactory, and spacer bars with check nuts were used against the chords in 1919. Then, for safety,

PLAN OF TOGGLE FOR NIAGARA ARCH, 1918 — 1919.



SECTION ON LINE *A-A*



ELEVATION

TOGGLE FOR
OPENING TOP CHORD JOINT *U8*

FIG. 16.

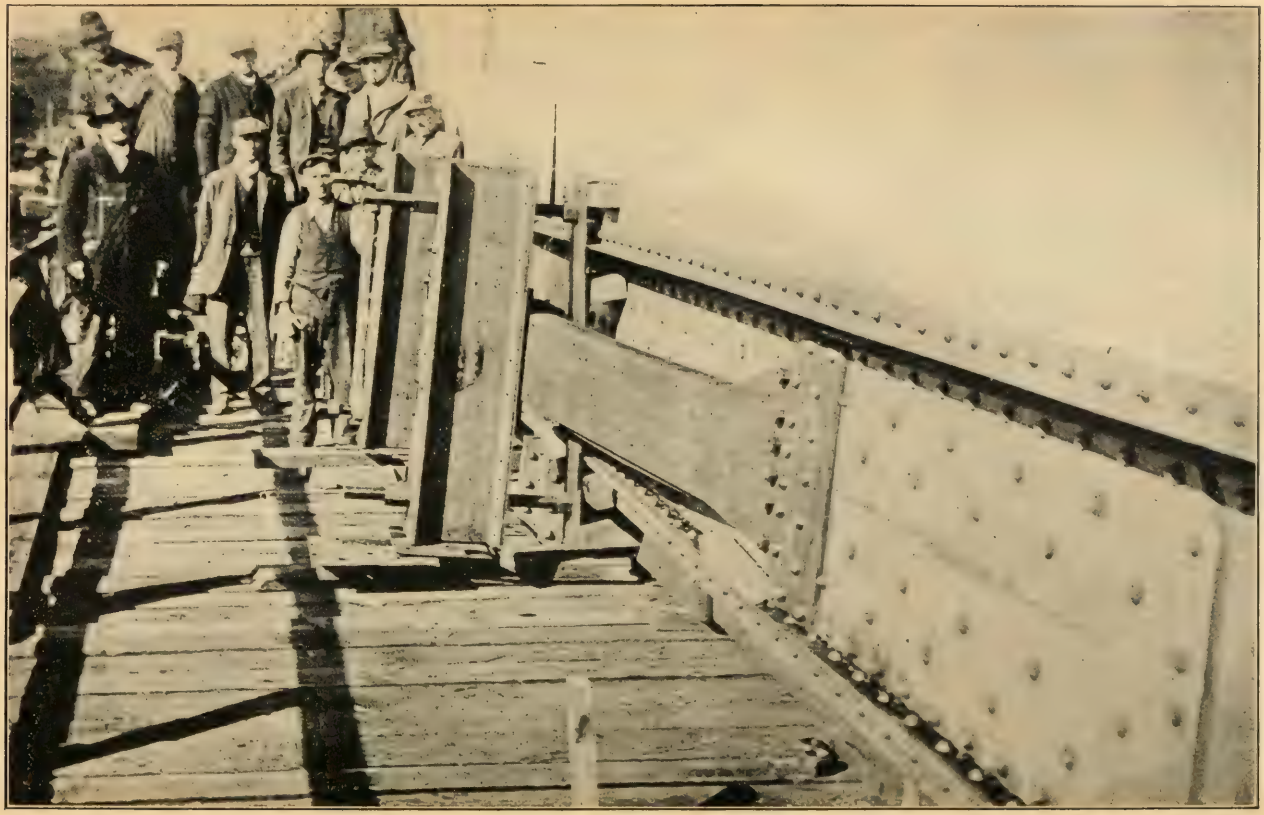


FIG. 17.—TOGGLE FOR ADJUSTING ARCH STRESSES, SOUTH TRUSS.

lock nuts were provided outside the main adjusting nuts on one pair of rods in each of the two toggles, and special long nuts were provided for the other sets of rods for making the final turns. Screw-jacks were also placed opposite the hips inside the chord, to account for the thrust from the spacing bars.

When the stress had been read for $U_7\ U_8$ in 1918, a dead load stress sheet (Table 5) was prepared showing the stresses from dead load only in each member of the arch ribs, for zero stress in the center top chord, and for other values advancing by steps of 200 000 lb. for each column in the table. From this it was possible to determine that it would be out of the question to change the center shim to make the arch truly three-hinged under dead load, and it was decided to prepare the new stress sheet for a center top chord dead load stress of 300 000 lb., and on the completion of all stress computations, this proved to be the value adopted for final use.

The old toggle was remodeled, a second one was made for the other chord, and after careful preparation the shims were changed on October 1st, 1919. Operating platforms were built outside and inside both chords (Fig. 17), about 2 ft. below the bottom flanges, so as to facilitate the erection and operation of the toggles. The rivets were cut out of all splices on both chords, on all beams and bracing in any way tying the two halves of the ribs together, and 50% of the $\frac{15}{16}$-in. holes were filled with $\frac{7}{8}$-in. bolts with washers on both sides, and these were only loosely tightened so as to allow the joints to slide the necessary distance on the gussets, the outside splice plates being entirely removed to permit changing the shims. The stringer connections were cut loose at the eight points at the U_7 floor-beams, thinner filler plates placed, and the connections bolted, 1-in. turned bolts being used in the connections under the north track carrying trains.

During the changing of the shims highway traffic was not stopped, but at 8.00 A. M. railroad traffic was entirely suspended. Two hours were consumed in completing bolting operations and adjusting the toggles, but at 10.00 A. M. the tightening of the screws began. The superintendent blew a whistle and each of the sixteen nuts, one on each end of each rod, was given four turns, and when all were abreast this operation was repeated. Some delays occurred, but on the completion of twenty-six turns in less than two hours, the old shims were loosened and could be withdrawn with the fingers. After lunch they

TABLE 5.—Dead Load Stresses in Arch for Various Values of Stress in U_7U_8.

All Stresses in Thousands of Pounds; — = Compression.

Member.	Value of Stress in U_7U_8.						
	0	200	400	600	800	1 000	1 200
Upper Chords							
U_0U_1	— 38	45	53	62	71	78	— 91
U_1U_2	— 55	72	96	116	138	156	— 179
U_2U_3	— 63	100	140	176	214	251	— 292
U_3U_4	— 60	121	182	242	307	368	— 432
U_4U_5	— 50	143	239	334	451	525	— 622
U_5U_6	— 35	169	306	442	580	718	— 855
U_6U_7	— 12	192	372	552	731	910	— 1 091
U_7U_8	— 0	200	400	600	800	1 000	— 1 200
Lower Chords							
L_0L_1	— 2 185	2 141	2 100	2 055	2 010	1 966	— 1 920
L_1L_2	— 2 057	2 008	1 953	1 900	1 846	1 793	— 1 739
L_2L_3	— 1 942	1 880	1 816	1 751	1 686	1 622	— 1 557
L_3L_4	— 1 855	1 775	1 696	1 616	1 532	1 453	— 1 370
L_4L_5	— 1 766	1 684	1 582	1 480	1 373	1 271	— 1 165
L_5L_6	— 1 742	1 611	1 479	1 344	1 207	1 075	— 940
L_6L_7	— 1 725	1 553	1 380	1 205	1 031	860	— 683
L_7L_8	— 1 728	1 512	1 298	1 084	867	654	— 438
Diagonals							
U_0L_1	123	149	177	205	235	263	293
U_1L_2	48	79	110	143	175	207	241
U_2L_3	22	56	94	130	168	203	242
U_3L_4	0	41	82	125	166	209	251
U_4L_5	— 15	34	81	130	180	227	278
U_5L_6	— 23	33	88	142	195	250	305
U_6L_7	— 25	28	79	129	180	232	283
U_7L_8	— 15	10	33	56	80	103	127
Verticals							
U_0L_0	— 460	486	512	539	567	593	— 620
U_1L_1	— 289	315	346	376	406	435	— 467
U_2L_2	— 230	261	293	325	358	390	— 423
U_3L_3	— 200	231	266	300	336	370	— 404
U_4L_4	— 175	210	244	279	314	350	— 385
U_5L_5	— 160	194	228	262	296	331	— 365
U_6L_6	— 158	186	213	240	268	295	— 324
U_7L_7	— 162	174	187	198	210	233	— 334
U_8L_8	— 284	....	....	....	....	....	— 284
Thrust, H	1 740	1 705	1 670	1 635	1 599	1 564	1 528
Reaction, V	1 782	1 782	1 782	1 782	1 782	1 782	1 782

were all removed and the new shims, about $\frac{3}{8}$ in. less in thickness, were placed, and on all adjustments being made, the nuts were slacked off five turns at a time, and by 4.00 P. M. the joints had all been bolted sufficiently to allow resumption of railway traffic. Not a mishap had occurred in completely jacking apart the top chords of the 550-ft. arch span, and in picking up with both toggles practically 700 tons. The adjustment of the center diagonals and the center sections of the bottom chord to the new stress conditions had been provided for by curving out the gussets between the diagonals and the center posts by means of a oxy-acetylene burning torch and by cutting out all the rivets in the connection at L_8 as far as possible with the south track out of commission; the number of rivets that were cut out was roughly 50 per cent.

The measurement of stresses in the toggles showed that at one time, near the maximum load, the small center slab was undergoing bending, due to the eccentric application of the load on the end. This slab was blocked and no trouble resulted.

The stresses, as read on October 1st, were calculated that evening and showed a wide variation from the 300 000 lb. total desired in each chord, as follows:

North chord	98 000 lb.
South chord	200 000 lb.

The check made of the old stress of 689 000 lb., as measured in 1918, showed for the north chord 715 000 lb. and for the south chord 688 900 lb., being only 3.8% larger for the north chord, presumably due to stress carried in 1918 by the four lines of stringers which were tightly fitted, and a practically identical reading for the south chord.

The large number of heavy freight trains that had been held up while the work was in progress, were run over the north track during the ensuing 16 hours, until final readings could be made the next morning, October 2d. Then the resulting total stress after full readjustment was as follows, adding for the stress that would be caused by the placing of the ties and rails of the south track which was just ready for laying:

North chord	288 000 lb.
Stress from south track	14 000 lb.
Total stress	302 100 lb.

South chord	255 000 lb.
Stress from south track	38 900 lb.
Total stress	293 900 lb.

These stresses may be considered as practically exact, that for the north chord being within 1% of the desired value, and that for the south chord within 2 per cent. This close agreement resulted from the reading of the dead load stress in 1918, the very careful calipering of the old shims, and the many check strain-gauge readings. Owing to the fact that the thickness of the new shims was reduced in exact proportion from that of the old shims, the distribution over the cross-section of the four chord sections, $U_7\ U_8$, remains practically the same as before the change.

Stress Sheet.

The recorded data on the former stress computations were very meager, consisting of the stress sheet* of the paper by Mr. L. L. Buck, a blue-print diagram of lateral and sway stresses, together with a record of the calculation for $\Sigma\ H$ for the arch ribs.

The original total stresses included simply live load, dead load, and temperature stresses, no account being taken of secondary stresses or impact. The unit stresses assumed were, for the diagonals in tension, dead load 20 000 lb. per sq. in., and live load 10 000 lb. per sq. in.; for compression in the top chords, dead load 18 000 lb. per sq. in., and live load

$$\frac{9\,000}{1 + \dfrac{\dfrac{9\,000}{l^2}}{30\,000\ r^2}};$$

for posts and diagonals in compression, dead load 16 500 lb. per sq. in., and live load

$$\frac{9\,000}{1 + \dfrac{\dfrac{9\,000}{l^2}}{30\,000\ r^2}}.$$

The resulting unit stresses on the sectional areas adopted were as follows:

Top chords	3 650 to 12 700 lb.
Bottom chords	11 500 to 12 800 lb.
Diagonals (tension)	10 600 to 12 600 lb.
Vertical posts	5 350 to 11 100 lb.

* *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 149, Plate XII.

The new stress sheet, Table 6, consists of a column for each class of stress, and by every method for each class which it was desired to have, either for comparison or final combination. The result was a sheet of thirty columns, affording much data in the way of valuable comparison and enlightenment for obtaining final maximum and minimum stresses. The first eight columns are all dead load or non-dynamic stresses, including dead load, temperature, snow and wind load stresses; the next nine columns are for train live load stresses both direct and reverse, together with impact by the Schneider formula, the Lindenthal formula, the proposed new American Railway Engineering Association formula added later, and from the Niagara impact readings with the strain gauge. Attention is called to the low values given by the proposed new A. R. E. A. formula, and their comparison with the Niagara readings, which are not necessarily a corroboration of the correctness of the proposed new A. R. E. A. formula. The comparison of the Schneider and Lindenthal values shows that there is not a wide divergence on the average for the compression members, but indicates much higher values by the Lindenthal formula for the tension diagonals, as would seem logical from the viewpoint of safety, although not corroborated by the Niagara strain-gauge results. The careful study of all the values would indicate that the results obtained by the proposed new A. R. E. A. formula are very low for the usual average speeds over main-line bridges. The fixing of the critical speed would therefore seem to be the first necessity of a perfect formula, and, secondly, the coefficient or function herein developed to be introduced for variation in speed—these two factors being intimately correlated.

The next seven columns of the stress sheet cover the highway live load, direct and reverse; the lateral live load; and the secondary stress percentage, with the equivalent in thousands of pounds, the percentages being 10% flat for the bottom chords, from 20 to 200% for the top chords, from 15 to 45% for diagonals, and from 30 to 60% for the vertical posts.

The last six columns comprise the total stresses as combined from the various stresses, giving both minimum and maximum stresses, and 50% of the reverse minimum. The last two columns give the allowable unit stresses as fixed for the revision of the structure and the maximum resulting from the combination of the co-existing actual stresses.

TABLE 6.—Stress Sheet.

All Stresses in Thousands of Pounds; — = Compression.

Dead Load, Temperature, Snow, and Wind Stresses.

Member.	Dead Load.			Temperature ±	Snow.	Wind.		
	True two-hinge.	Existing shims.	Modified shims.			Plane of truss.	Lateral and sway.	Total.
Upper Chords								
U_0U_1	— 59	65	49	20	— 1	— 89	0	— 89
U_1U_2	— 111	124	84	50	— 2	— 160	81	— 79
U_2U_3	— 169	189	120	90	— 4	— 210	148	— 62
U_3U_4	— 233	264	151	148	— 6	— 223	201	— 22
U_4U_5	— 318	367	191	229	— 8	— 193	243	50
U_5U_6	— 422	490	238	332	— 12	— 110	273	163
U_6U_7	— 527	614	282	434	— 15	— 2	294	292
U_7U_8	— 572	689	300	480	— 17	56	305	361
Lower Chords								
L_0L_1	— 2 063	2 040	2 120	107	— 59	— 898	151	— 747
L_1L_2	— 1 910	1 881	1 980	127	— 54	— 744	447	— 297
L_2L_3	— 1 764	1 729	1 848	155	— 51	— 628	731	103
L_3L_4	— 1 628	1 587	1 735	193	— 48	— 550	995	445
L_4L_5	— 1 497	1 442	1 633	248	— 44	— 517	1 229	712
L_5L_6	— 1 367	1 297	1 545	324	— 39	— 532	1 424	892
L_6L_7	— 1 230	1 145	1 467	421	— 34	— 608	1 565	957
L_7L_8	— 1 113	1 009	1 405	519	— 29	— 707	1 635	928
Diagonals								
U_0L_1	198	251	163	68	3	300	....	300
U_1L_2	133	154	95	77	3	191	....	191
U_2L_3	122	143	75	87	3	100	....	100
U_3L_4	117	139	61	100	4	23	....	23
U_4L_5	121	147	58	117	4	— 47	....	— 47
U_5L_6	133	160	60	132	5	— 103	.. .	— 103
U_6L_7	121	147	54	121	5	— 126	....	— 126
U_7L_8	54	64	22	54	2	— 69	....	— 69
Verticals								
U_0L_0	— 532	549	499	65	— 9	— 284	— 388	— 672
U_1L_1	— 370	386	330	71	— 8	— 177	— 77	— 254
U_2L_2	— 318	336	277	77	— 8	— 88	— 63	— 151
U_3L_3	— 294	312	248	82	— 9	— 19	— 48	— 67
U_4L_4	— 273	291	227	85	— 8	32	— 35	— 3
U_5L_5	— 256	274	211	82	— 8	67	— 25	42
U_6L_6	— 236	249	200	65	— 8	68	— 18	50
U_7L_7	— 197	202	180	27	— 6	37	— 14	23
U_8L_8	— 100	— 100	— 100	...	— 4	.....	— 11	— 11

TABLE 6.—Stress Sheet (*Continued*).

Live Load Stresses.

Member.	Railway.		Highway.		Lateral Live Load.			Secondary.	
	Direct.	Reverse.	Direct.	Reverse.	Plane of truss.	Lateral and sway.	Total.	Percentage of railway direct.	Equivalent stress.
Upper Chords									
U_0U_1....	— 185	117	— 19	14	— 7.1	0	— 7.1	200	370
U_1U_2....	— 383	251	— 41	29	— 13.2	11.1	— 2.1	100	380
U_2U_3....	— 610	380	— 66	44	— 17.5	20.7	3.2	55	330
U_3U_4....	— 885	522	— 93	59	— 19.5	28.8	9.3	35	290
U_4U_5....	— 1 025	580	— 114	66	— 18.4	35.5	17.1	25	250
U_5U_6....	— 1 205	585	— 134	66	— 14.8	40.6	25.8	25	300
U_6U_7....	— 1 120	387	— 129	42	— 9.6	44.3	34.7	30	340
U_7U_8....	— 915	160	— 105	18	— 6.2	46.6	40.4	20	180
Lower Chords									
L_0L_1....	— 2 505	....	— 292	....	— 91.3	12.4	— 78.9	10	250
L_1L_2....	— 2 370	32	— 273	3	— 78.0	37.2	— 40.8	10	240
L_2L_3....	— 2 270	91	— 260	9	— 68.0	61.7	— 6.3	10	230
L_3L_4....	— 2 220	176	— 251	17	— 60.2	85.3	25.1	10	220
L_4L_5....	— 2 150	278	— 240	28	— 56.1	106.7	50.6	10	210
L_5L_6....	— 2 010	354	— 223	36	— 55.4	124.4	69.0	10	200
L_6L_7....	— 1 785	336	— 194	35	— 58.1	137.1	79.0	10	180
L_7L_8....	— 1 490	228	— 158	24	— 62.8	144.2	81.4	10	150
Diagonals									
U_0L_1....	618	— 398	63	— 46	23.7	.....	23.7	35	220
U_1L_2....	573	— 358	58	— 42	16.6	.....	16.6	45	260
U_2L_3....	535	— 301	53	— 36	8.9	.....	8.9	35	190
U_3L_4....	514	— 248	52	— 30	3.2	.....	3.2	20	100
U_4L_5....	518	— 220	52	— 26	— 1.3	.....	— 1.3	15	80
U_5L_6....	565	— 268	58	— 30	— 5.0	.....	— 5.0	15	80
U_6L_7....	736	— 537	81	— 55	— 6.0	.....	— 6.0	20	150
U_7L_8....	940	— 850	102	— 90	— 4.0	.....	— 4.0	15	140
Verticals									
U_0L_0....	— 857	380	— 86	44	— 22.6	— 29.8	— 52.4	40	340
U_1L_1....	— 770	325	— 80	39	— 15.3	— 9.0	— 24.3	40	310
U_2L_2....	— 702	268	— 73	31	— 7.9	— 8.0	— 15.9	40	280
U_3L_3....	— 654	200	— 67	24	— 2.9	— 6.9	— 9.8	35	230
U_4L_4....	— 596	141	— 61	17	1.0	— 5.9	— 4.9	30	180
U_5L_5....	— 550	131	— 58	14	3.1	— 5.0	— 1.9	35	190
U_6L_6....	— 540	221	— 62	22	3.2	— 4.3	— 1.1	60	320
U_7L_7....	— 611	338	— 67	35	2.0	— 3.8	— 1.8	45	280
U_8L_8....	— 288	....	.....	....	0	— 3.6	— 3.6	..	...

TABLE 6.—Stress Sheet (*Continued*).

Impact Stresses.

Member.	Railway Impact, Direct.				Railway Impact, Reverse.			
	Niagara.	Schneider.	Lindenthal.	New A.R.E.A	Niagara.	A.R.E.A.	Lindenthal.	New A.R.E.A.
Upper Chords								
U_0U_1	— 55	— 109	— 150	— 76	28	55	61	23
U_1U_2	— 111	— 222	— 306	— 149	60	120	139	53
U_2U_3	— 174	— 348	— 473	— 220	93	186	217	87
U_3U_4	— 229	— 458	— 622	— 275	131	261	308	130
U_4U_5	— 272	— 544	— 715	— 298	151	302	334	162
U_5U_6	— 302	— 603	— 760	— 301	161	322	323	193
U_6U_7	— 269	— 538	— 604	— 236	112	224	170	151
U_7U_8	— 238	— 476	— 504	— 258	43	85	32	45
Lower Chords								
L_0L_1	— 451	— 902	— 788	— 225	....	....	....	....
L_1L_2	— 450	— 909	— 806	— 261	14	27	2	29
L_2L_3	— 455	— 910	— 832	— 295	34	68	5	68
L_3L_4	— 467	— 933	— 883	— 333	60	120	23	105
L_4L_5	— 473	— 946	— 915	— 366	89	178	53	139
L_5L_6	— 453	— 926	— 912	— 402	106	212	80	152
L_6L_7	— 438	— 876	— 880	— 428	94	188	72	114
L_7L_8	— 388	— 775	— 765	— 417	61	122	35	666
Diagonals								
U_0L_1	185	370	512	266	— 92	— 184	— 205	— 80
U_1L_2	175	350	521	258	— 81	— 161	— 196	— 68
U_2L_3	169	337	502	268	— 66	— 132	— 155	— 51
U_3L_4	165	329	502	257	— 55	— 109	— 122	— 42
U_4L_5	169	337	525	285	— 48	— 95	— 102	— 35
U_5L_6	170	339	500	243	— 62	— 123	— 138	— 54
U_6L_7	177	353	500	155	— 156	— 312	— 453	— 210
U_7L_8	240	480	760	244	— 230	— 459	— 745	— 264
Verticals								
U_0L_0	— 257	— 514	— 575	— 368	88	175	123	76
U_1L_1	— 223	— 446	— 537	— 300	81	161	128	70
U_2L_2	— 207	— 414	— 513	— 288	63	126	96	54
U_3L_3	— 196	— 392	— 493	— 281	46	92	63	40
U_4L_4	— 179	— 358	— 447	— 256	33	65	37	28
U_5L_5	— 154	— 308	— 370	— 187	32	63	37	32
U_6L_6	— 124	— 248	— 281	— 103	68	135	120	100
U_7L_7	— 147	— 294	— 371	— 135	97	193	217	128
U_8L_8	— 120	— 240	— 412	— 250	....	....	....	....

TABLE 6.—Stress Sheet (*Continued*).

Maximum, Minimum, and Unit Stresses.

Member.	Maximum.	Minimum.	50% of reverse minimum.	Area, in square inches.	Unit Stresses.	
					Maximum.	Allowed.
Upper Chords						
U_0U_1	— 691	120	60	69.5	9.9	18.2
U_1U_2	— 1 039	279	140	69.5	14.9	18.2
U_2U_3	— 1 383	438	219	79.6	17.3	18.2
U_3U_4	— 1 749	627	314	103.5	16.1	18.2
U_4U_5	— 2 114	709	354	138.4	15.3	18.2
U_5U_6	— 2 585	722	361	167.1	15.5	18.3
U_6U_7	— 2 722	448	224	176.5	15.5	18.3
U_7U_8	— 2 417	129	65	175.9	13.7	18.3
Lower Chords						
L_0L_1	— 5 477	— 1 956	.....	316.7	17.3	19.6
L_1L_2	— 5 194	— 1 734	.....	289.5	17.9	19.7
L_2L_3	— 4 955	— 1 475	.....	282.0	17.6	19.7
L_3L_4	— 4 807	1 182	.....	270.0	17.6	19.8
L_4L_5	— 4 642	— 854	.....	262.4	17.7	19.8
L_5L_6	— 4 426	— 547	.....	249.3	17.8	19.9
L_6L_7	— 4 107	— 344	.....	225.8	17.9	19.9
L_7L_8	— 3 697	— 281	.....	191.1	19.3	19.9
Diagonals						
U_0L_1	1 293	— 406	— 203	72.3	17.8	20.0
U_1L_2	1 221	— 425	— 213	67.3	18.2	20.0
U_2L_3	1 106	— 368	— 184	61.1	18.1	20.0
U_3L_4	1 001	— 316	— 158	56.8	17.6	20.0
U_4L_5	1 009	— 380	— 145	56.8	17.8	20.0
U_5L_6	1 087	— 359	— 180	64.9	16.8	20.0
U_6L_7	1 691	— 748	— 374	94.1	18.0	20.0
U_7L_8	2 023	— 1 170	— 585	126.4	16.0	20.0
Verticals						
U_0L_0	— 2 060	45	23	123.4	16.7	16.8
U_1L_1	— 1 755	156	78	97.1	18.1	16.5
U_2L_2	— 1 595	121	61	91.1	17.5	16.5
U_3L_3	— 1 466	58	29	84.6	17.3	16.5
U_4L_4	— 1 322	3	2	78.7	16.8	16.5
U_5L_5	— 1 243	3	2	74.7	16.6	18.0
U_6L_6	— 1 301	140	70	74.7	17.4	19.0
U_7L_7	— 1 274	300	150	75.2	17.0	19.7
U_8L_8	— 484	100	....	41.9	11.5	19.4

Although the posts from U_1 L_1 to U_4 L_4 show an excess, a consideration of the secondary stress strain-gauge readings shows them to be within the limits set for direct compressive stress; furthermore, the effective length is much reduced by the highway floor-beams and stringers connecting at the highway level.

Methods of Reinforcing.

The reinforcement of the posts of the 115-ft. span, and of the viaduct, consisted of two angles back to back on each web, but in some cases a filler plate was used. The holes in these plates and angles matched the old rivet spacing in the webs as far as it extended, and when the new material had been bolted in place by a few bolts, the additional holes were drilled through the webs with high-speed air drills. Every other hole was bolted before riveting, and the bolts tightened before riveting but after the dead load had been relieved, either by the toggle on the 115-ft. span posts as already described, or by shores and jacks to carry the dead and live load for the approach columns.

The old cover-plates on the approach girders, the stringers, and the floor-beams of the spans were cut loose by cutting out the rivets by hand, except for the old rusted top plates which were to be thrown away and could stand the heat from a cutting torch. These rivet heads were burned off at the rate of 600 to 700 per day, or equal to the number removed by four hand gangs, and the rivets were then backed out by hand. The new top cover-plates were countersunk so as to leave a smooth surface to receive the 14-in. maple risers, but experience indicates the desirability of countersinking top flange rivets in all railway bridge stringers or deck girders, so that ties need not be bored or notched for rivet heads. All new plates were riveted up after removing the dead load stress by shoring up from below with 8 by 8-in. shores and wedges for the stringers on the spans, one for each stringer; but two 12 by 12-in. shores were used under each of the approach girders, which were from 40 to 52 ft. in length.

The reinforcing plates on the curved brackets were clamped in place, the holes drilled through the old flange and the riveting done without any shoring, as there was practically no dead load stress to be considered. The new counters in the 115-ft. spans were $1\frac{3}{16}$-in. square U-bars, or two bars replacing one of the same size, which would

have been stressed to more than 40 000 lb. under E-60 loading. The old loop eyes were burned off on each side of the pin by using a torch, so that the old counters were easily removed, after previously putting in four strands of wire rope, tightened by turnbuckles to carry the counter stress until the new rods were in place. The short U-bars were then slipped around the top and bottom pins, the straight sections connected up, and tightened with the four turnbuckles to an initial stress of 3 000 lb. per sq. in., as shown by strain-gauge readings.

The viaduct posts had originally been built with bases 40 by 43 in., resting on pedestal blocks of stone 5 by 5 by 2 ft., entirely above the pavement. This had become an obstruction to traffic of a very serious nature, and as two of the stone pedestals had become cracked, it was decided to replace the two nearest each entrance to the highway floor with extensions of the steel post section to below pavement level, with a flaring base to distribute the pressure over the concrete footing. These bases were built up of channels and plates, as it was impossible to obtain steel castings within any reasonable time. They were fully spliced to the columns after the old base plates and connections had been burned off, the pockets having been filled with water-proof concrete directly against the metal. The space between the bases and the pavement was filled with concrete in concave curves to ward off the wheels of vehicles. The additional gain in roadway width was about 4 ft. The post reinforcing was increased to provide for the increased value of $\frac{l}{r}$.

Sway Bracing.

The new diagonal sway bracing (Fig. 18), in the bents, U_0L_0, U_2L_2, U_4L_4, is composed of two 12-in. channels for each member, laced with 3 by 3 by $\frac{3}{8}$-in. angles double-riveted, and having a stiffening angle on each tie-plate. The old rods were not removed until four wire ropes with turnbuckles had been placed to take the stress, one vertical panel being replaced at a time, and it was required that a panel be finished the day it was begun, as a measure of safety. The connections to the arch ribs was made by means of the large gussets connecting generally to old holes with very few new ones to drill. The posts were also diaphragmed opposite the bottom gussets. The center gussets were made large enough and holes were provided for connecting struts that will be necessary when two more ribs are

added to the arch, and then the end connections can be increased in strength by placing the rivets in double shear. When new ribs are added, stiff bracing will be placed in the alternate bents as well. The bracing of the end bents, U_0L_0, has connected to it steel stairways as shown by Fig. 18, the outside stringer being a single channel, and the steps formed of two rods with pipe separators. The rise makes the stairways very easy to climb and they are a distinct advantage in ease of access for making repairs to the bridge.

The two vibration struts in each bent, U_1L_1, and one in each bent, U_3L_3, as shown in Fig. 18, are 10 ft. deep, with two 5 by 3 by $\frac{3}{8}$-in. angle flanges, and $4\frac{1}{2}$ by 3 by $\frac{3}{8}$-in. angle web members. They are purely for counteracting vibration, and are connected or steadied by angle lugs at the middle pin of the sway rods. They, as well as the diagonals, serve the purpose perfectly, and the side sway and swing of the bridge which was so objectionable has been practically eliminated, besides making the arch perfectly safe against vibration from heavy trains.

New Grade Line.

The grade line of the arch was originally made to conform to the large camber of the old suspension bridge, as a means of erecting the arch around the stiffening trusses of the latter without stopping traffic. This made a sag in the grade of practically 18 in. between the center of the arch span and the New York Central Railroad crossing on the American shore, or practically at the shore end of the 115-ft. span. The heavy grade incidental to this made it necessary for the heavier freight trains to cross the bridge at speeds of from 20 to 30 miles per hour, which were considered unsafe for a bridge of this character—so high and directly above the Rapids. Therefore, it was decided to remove this sag by means of steel risers on top of the stringers, which also were made to serve as the new top flange of these stringers on the American half of the arch and on the 115-ft. span. The rise in the first panel next to the center of the arch was made entirely in the 14-in. maple risers of the new floor.

The risers in the next two panels consist of four angles telescoped, and, for the remainder of the arch, of four angles and a web-plate, with stiffeners every 4 ft., all risers being tapered to conform to the new grade. The risers on the 115-ft. span are of Bethlehem H-beams, 10 and 12 in. deep, with end stiffeners only.

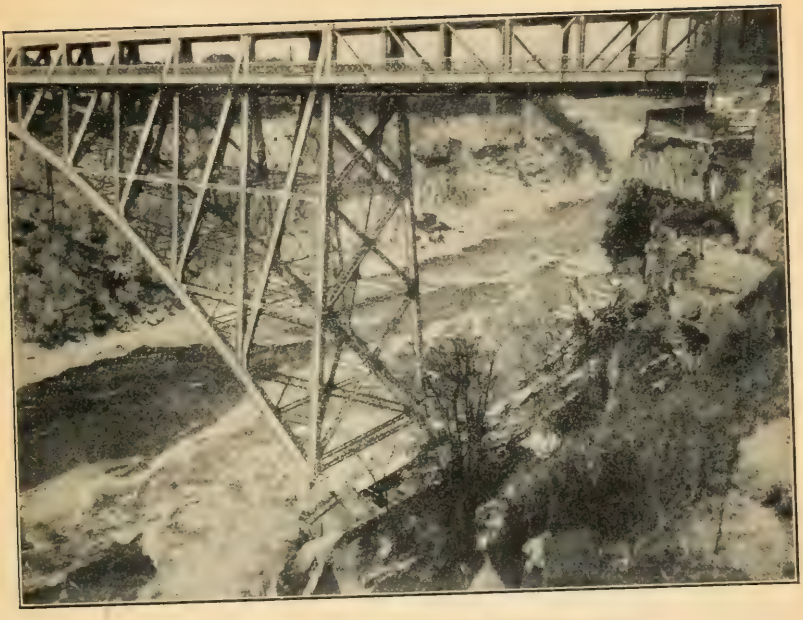


FIG. 18.—NEW SWAY AND VIBRATION BRACING.

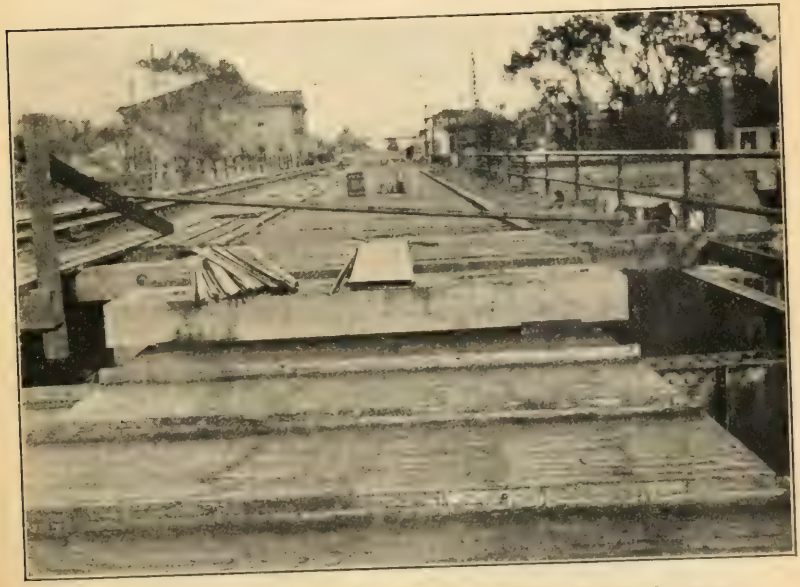


FIG. 19.—COPPER-STEEL RAILWAY PROTECTION FLOOR, SHOWING TIE RISERS AND COVER STRIPS.

The American approach was jacked up bodily 12 in., and cast-steel bases were inserted between the steel bases of the columns and the masonry. This jacking was done for all three of the American 48 ft. 6-in. double-track spans, during the two hours when the trains were stopped. Bents of 12 by 12-in. timbers had been previously placed, and eight 60-ton jacks on shores, two at each point, were used to raise the 300 tons. The jacking began on a whistle signal, and all points were raised and blocked 1 in. When all were abreast, a second signal was given for raising the second inch. Marks had been made to watch for creeping, but none occurred. With the new castings and sheet lead in place, the anchor-bolts were temporarily placed, the new north track was connected, and trains were allowed to cross.

New Railway Floor.

The old railway deck had been placed originally with galvanized strips between the ties, small wooden side gutters, and 2½-in. galvanized downspouts to protect the steelwork from salt water drip and the highway traffic from water coming through from above. This proved unsatisfactory, and, later, wooden strips with a groove for a gutter were used between the ties. This was apparently a passable protection, but in the dry weather period a great deal of leakage made it unpleasant for the highway traffic. While making the 1918 inspection, the condition of the top flanges and angle laterals between them, of the stringers, floor-beams, and girders was investigated to ascertain the extent of the salt-water corrosion from refrigerator car drippings. The only places that could be reached were the small spaces between the ties, and below the small, wooden gutter strips. It was found that the corroded metal could easily be pried off in flakes from $\frac{1}{8}$ to $\frac{3}{8}$ in. thick. Later, by using a rolling staging, spots were scraped on each top flange at frequent intervals and it was found that plates and angles originally from $\frac{3}{8}$ to $\frac{1}{2}$ in. thick, were wasted away on an average of 50%, or in extreme cases only $\frac{1}{16}$ to $\frac{1}{8}$ in. thickness remained.

Studies were at once begun to find some type of floor that would be a practically perfect protection against this salt water and also the blow-off water from locomotive boilers. There was found to have been no real attempt by any one to solve the problem, except the endeavor of the American Railway Engineering Association to have

efficient drip tanks placed in refrigerator cars, and the use in some cases of heavy concrete troughs to carry ballast and tracks.

The latter solution is represented by the heavy reinforced concrete floor as designed and used by Charles F. Loweth, M. Am. Soc. C. E., Chief Engineer of the Chicago, Milwaukee and St. Paul Railway. This is a splendid type of construction, but as it weighs 3 400 lb. per lin. ft. of track, could not be considered for use on the Arch Bridge. One of the lightest floors of this type is in use by the Buffalo, Rochester and Pittsburgh Railway, and was only slightly too heavy to be considered, weighing 2 200 lb. per lin. ft. of single track. The design shown in cross-section in Fig. 20 was within the weight limit desired, and a test section was made with the cement gun. Tested with its own

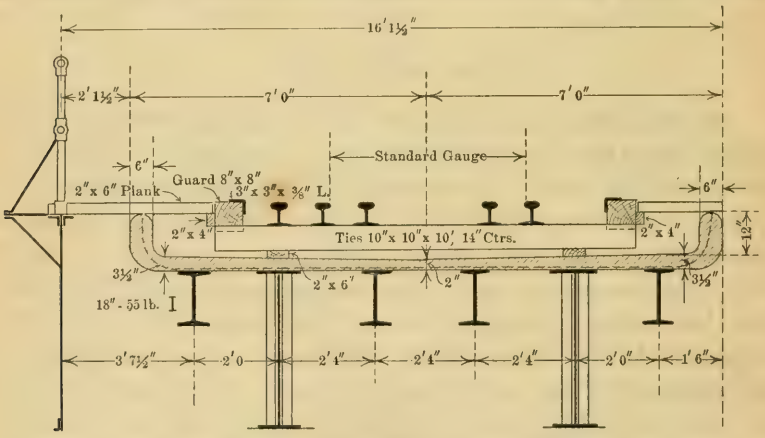


FIG. 20.—LIGHT CONCRETE FLOOR AS TESTED.

weight over a span of 13 ft. on 12-in. blocking, it sagged about 6 in. and developed hair cracks. It could be tetered up and down 6 in. with one's foot and return to its original sagged elevation, and so far as tests indicated would be satisfactory for the short spans between stringers and new jack stringers. It was discarded mainly on account of the certainty that it would shatter under traffic. The idea, with certain evident modifications, is worthy of a trial on some new bridge.

The design, which was finally approved from an engineering standpoint, was a ⅜-in. flat steel plate protection with proper protective coating. Furthermore, this made it possible to carry the water away by side downspouts, but when the cost was considered, and the cost of maintenance as well, some lighter plate floor was found desirable.

The suggestion of the use of $\frac{1}{8}$-in. special iron was at once rejected, and a plan for $\frac{3}{16}$-in. special iron floor was prepared for bids. The labor and shop conditions were finally responsible for the use of this $\frac{3}{16}$-in. floor, shown in detail in Fig. 21, which could be largely fabri-

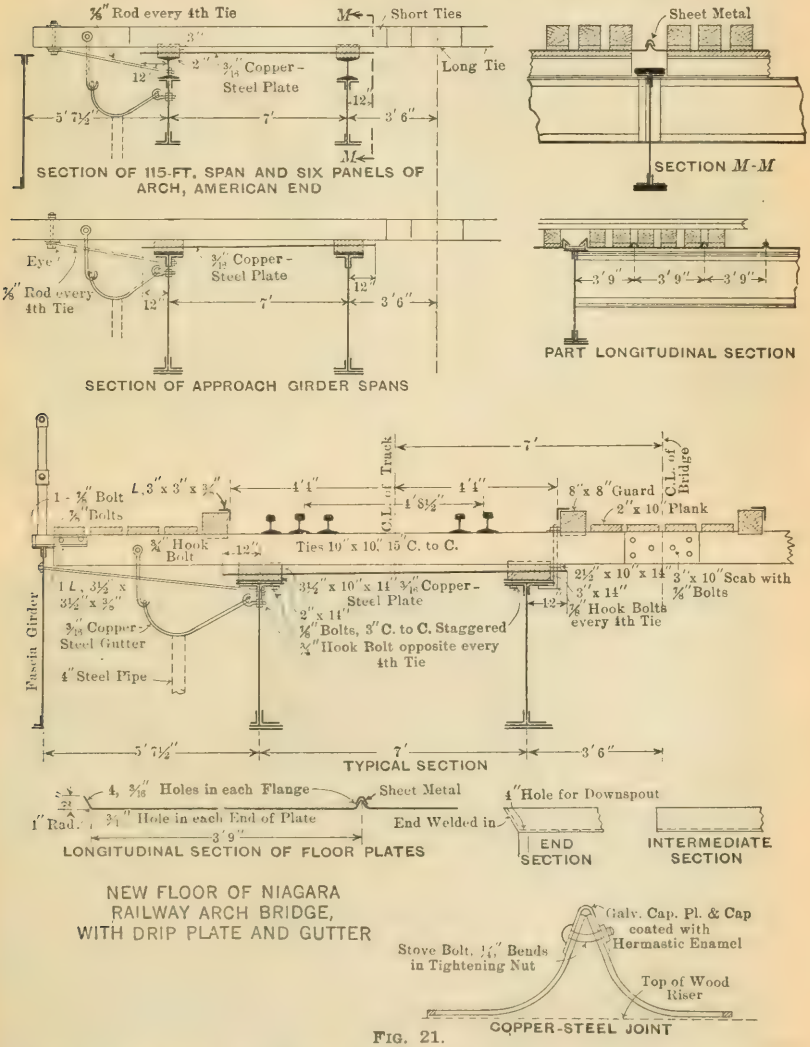


FIG. 21.

cated in the field, and easily maintained and repaired by ordinary workmen. The bids were received both on the special iron and copper steel, and the latter was adopted on account of a material saving. No

idea was entertained, however, that any metal as usually painted would last long when undergoing the corrosive action of the salt-water brine; therefore, the copper steel was coated, after fabrication and laying, with Hermastic enamel of a grade such as is used for the double bottoms of ships. The metal was first carefully cleaned by wire brushing and rubbing with burlap and coarse rags, coated with the special primer, and then the Hermastic enamel applied "piping" hot. Even then it could only be given one spread of from 12 to 18 in. with the special daubers used, and on account of the thin metal being kept constantly cool by the current of air up and down the Gorge, about 50% more enamel was used than was expected, giving a coating more than $\frac{1}{4}$ in. in average thickness.

The design is quite plainly shown in Figs. 19 and 21, but attention is called to a few vital points. The 9-ft. floor-plates have a fall of 1 in 84, arranged by the inside and outside 14-in. maple stringer risers, thus draining the water to the large gutters on each side of the bridge. This side slope is taken out of the track by making the outside 10 by 14-in. hardwood tie block 1 in. thicker than the inside one. These blocks are dapped into the 10 by 10-in. ties about $\frac{1}{2}$ in., the ties being dapped to a uniform thickness of $9\frac{1}{2}$ in. The blocks are spiked edgewise to prevent splitting, are fastened to the ties with two 6-in. spikes, and are treated with hot carbolineum, as are also the stringer risers.

The copper-steel floor sections are flanged, as shown, to a width of 3 ft. 9 in., so that a section can be pulled out sidewise for re-coating or repair, by slightly jacking up the track. The flanges are protected on the edges by a cap of No. 16 galvanized steel, coated with enamel and held in place by $\frac{5}{16}$-in. stove bolts so as to be easily renewed. When the plates were laid in place on the 14-in. maple risers, with a clear span of 5 ft. 10 in., they were perfectly stiff for carrying the workmen, and the writer, who weighs 200 lb., was unable to sag them by jumping up and down in the middle of a sheet. Some few of the plates were dished in flanging and required jacking up to level.

No injury was done the coating by walking over it, and it is not expected that it will be affected by the 120° variation in temperature from summer to winter. Cleaning it with a scraper would be injurious, and brushes are used, ordinarily, with frequent flushing by a hose.

The flanging of the plates was done at the bridge by the home-made bending machine shown in Fig. 22. This was constructed from old 12 by 12-in. timbers drift-bolted together with the edges of the jaw

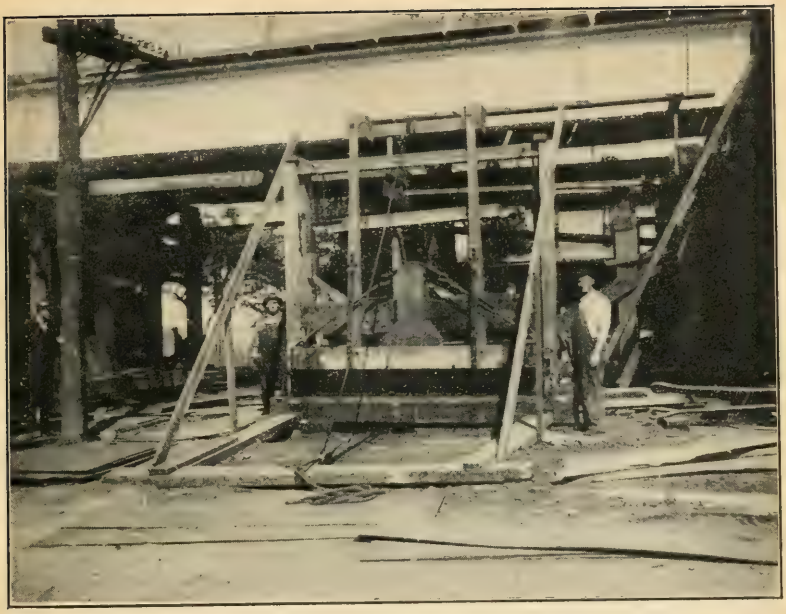


FIG. 22.—BENDING MACHINE FOR COPPER-STEEL FLOOR PLATES.

FIG. 23.—SKEWBACK THRUST WALL AND ARMORED CASING, CANADIAN SIDE.

timbers protected by old angles and plates. The copper-steel plates were extraordinarily stiff, and the flanging timber was trussed by a hog rod as shown. This timber was operated by two oak levers about 10 ft. long and by lines running over snatch blocks to the nigger-heads of a hoist engine. The plates were clamped in the machine by the counterweighted timber, which in turn was clamped down by the two levers bound on the edges with strap iron, which turned down on to plates spiked to the timbers. The exact width of the plates was obtained by using gauge angles on the bed of the machine, and the exact angle of the bend was first found by trial, allowing for the spring of the plates. The machine was operated without trouble to bend about 600 plates, five men bending an average of about 50 to 60 plates per day of 8 hours. These plates under the tracks have wooden stop-waters of 3 by 3-in. carbolineum-coated timber, spiked to the under side of the ties so as to bear tightly against the inner ends of the floor-plates.

The gutters (Fig. 21) were formed into the desired cross-section by bending rolls at the shop, after being punched on the interior ends for lap splicing with $\frac{3}{8}$-in. rivets, which were driven by air guns in the field. The ends were welded in the field with a welding torch, and the holes for the 4-in. downspouts were burned in the field with a torch.

The gutters have a fall of 6 in. for the girder spans, varying from 40 ft. to 52 ft. 3 in. for the 19 ft. 2-in. panels, and 4 in. for the arch panels of 34 ft. 4½ in. length. This is a slope of from 1 in 100 to 1 in 80, and gives quick drainage. The large size of the gutters was adopted in order to protect the highway deck near the sidewalks from splash and drip beyond the floor-plates.

The downspouts are of 4-in. steel pipe with screw joints, but attached to the gutter with flanges having oak filler gaskets soaked in linseed oil, and band-sawed to fit the curve of the gutter. All the downspouts are long enough to discharge below the bottom chords of the three spans, and are supported below by band clamps on the pipe and fastened to convenient lateral gussets.

The track is fastened down by hook-bolts to the inside stringers at every fourth tie, and anchored sidewise by the eye-bolt rods on the approach girders, and in the same manner on the spans, where the raised track is above the fascia girders. On the remainder of the spans the fascia girders have tie-rods to the stringers, as shown, with angle clips connecting every fourth tie to the top flange of the fascia girder.

The cost of repairs to make good the damages caused by salt water was over $30 000 and the cost of the copper-steel salt-water protection was about the same. These two items indicate how serious a problem is confronted in the matter of having refrigerator cars equipped with salt-water drip-tanks. These might be made very easily and economically by using large wooden boxes of matched flooring and lining them with thin sheet copper, well nailed in place to prevent its being stolen. These tanks cannot be emptied automatically, since they would be about as likely to discharge on a bridge as elsewhere; so they must be arranged for quick discharging, and yardmen must be trusted to keep them emptied.

The track as finished has a practically smooth surface and only about 0.1% grade. It belongs to and was laid by the railway company, lessees of the railway deck, and is of the best type of construction throughout, as was mutually agreed upon. The rails are 100 lb. per yd., of the A. R. E. A. section, with continuous bridge joints. The tie-plates, 7 by 10½ in., are of the Lundie type, especially large and heavy. The rails are laid with tight joints to prevent hammering and are provided with large expansion joints in each track of the Quebec Bridge type, at both of the ends of the bridge, to allow for approximately 5 in. of expansion, which is concentrated at the shore ends of the 115-ft. spans. Inside guard-rails of 100-lb. section are used, and an outside steel guard-rail of the same size spaced 5 ft. from the inside of the inside track rail to the inside of the guard. Borden tie-spacers are used between the ties, two lines to each track. The walkways are of 3 by 3-in. timbers laid ½ in. apart.

The old floor was very noisy during the passing of trains, but the new one, due to the maple risers and the Hermastic enamel, is very quiet and as nearly noiseless as is possible to build. Certainly, it is a demonstration that elevated railways of practically noiseless construction can be built. The cost of the high-class tracks alone was about $12.90 per lin. ft. of one track, and including the copper-steel floor complete about $27.50 per lin. ft. of one track. This would indicate that no bridge on account of extra cost need be constructed, leaving the upper metal work exposed to corrosion, and the floor system and bridge subject to destructive local impact increments, due to the desire for the possible financial saving that would result from the use of a less perfect track, and the lightest possible protection.

Highway Floor.

The new highway floor was built for a capacity of 2 130 lb. per lin. ft., on the trusses of the 115-ft. spans, and of 1 500 lb. per lin. ft. of bridge on the trusses of the arch span. The joists on the roadway are 9-in. 18-lb. I-beams, spaced about 2 ft. 4½ in. on centers, and calculated for a live load of 100 lb. per sq. ft. of roadway and for the concentrated load from a 15-ton motor truck. The sidewalk joists are 6-in. 15-lb. I-beams, calculated for a live load of 80 lb. per sq. ft. of sidewalk.

The live load capacities were assumed after a careful consideration of the data gathered by the late C. C. Schneider, Past-President, Am. Soc. C. E., and in the investigation of the Blackwell's Island Bridge in New York City.

The joists were fastened to the bridge stringers by 3 by 3 by ⅜-in. clips, bolted through the joist flange only. The 2½-in. hard pine sidewalk floor and the 3½-in. white oak roadway plank are fully spiked to pine spiking strips, which had been treated with hot carbolineum, and which are bolted to the top flanges of the steel joist. The curb was set back 4½ in. on each side of the road to give more roadway room, making the clear width 28 ft. 9 in. The plank was spiked with full ¼-in. openings to give proper drainage, and the spiking strip for carrying the ends of the sidewalk plank was spiked to a raising block 3 in. thick every 3 ft. to give plenty of opening for the dirt to be blown and scraped into the river. A protection of 3 by ¼-in. strap iron was spiked at the curb to the ends of the sidewalk plank, which had been previously treated against decay.

Highway Entrances.

The bridge when rebuilt in 1896 had a comparatively light highway traffic, and no need was found to give the greatest room possible for pedestrians and vehicles. The change made in the viaduct column bases to give about 4 ft. more roadway directly at the highway entrance has been described, and the fact noted that all the stone pedestals that were allowed to remain were cut to a batter to allow more hub room in the roadway, and they were curved out at the sidewalks to give more room for pedestrians. The width of the walks was further increased by cutting off the toll windows about 15 in., also the corner of the American toll-house to give about 20 in. more width. The

corner of the building was supported as an eccentric load on a special steel column and the space on each side turned into windows to give better light in the Customs Office.

The sidewalk to Whirlpool Street on the American side, about one and one-half blocks, was changed and landscaped with a curved walk to the north side of Mill Street, so as to avoid crossing the vehicle traffic twice. This change and a similar one across Mill Street made it possible very materially to add to the width of this street and to carry it practically full width to the bridge entrance. The railings to the small parks on both shores were improved, and reinforced concrete fence posts of a simple but neat design were used at the Canadian park. The same high standard as applied to the bridge itself was used in putting all the accessories in first-class condition to care for the present extensive highway traffic and to provide for future reasonable increase.

Sand-Blasting and Painting.

The paint on the structure was in very good condition, except as already detailed in regard to the upper work of the railway deck. It had been completely renewed three times in the preceding 22 years, with a 75% home-mixed white lead, zinc, and linseed oil paint. While no comprehensive method had been adopted in removing rust spots before applying the new coats of paint, the deterioration from rusting was practically nothing (except on the railway floor) on starting to clean the structure thoroughly with Mott sand-blast machines, and it was found that the original coats of red lead and white lead and zinc were intact over about 80% of the 480 000 sq. ft. of surface. Therefore, the sand-blasting was only used to clean off all dirt, loose and dead paint, and thoroughly to burrow out the small rust spots. As soon as possible after cleaning, these rust spots were coated with Tockolith to kill any small amount of rust missed, or the starting of incipient rust. The machines used for the sand-blasting were two No. 1 Mott sand-blasting machines and one No. 2.

When it was found that silica sand would cost more than $6 per ton delivered at the bridge from Illinois, several local sands were tried, and the dried locomotive sand procured from the Grand Trunk Railway roundhouse was found to be sharp enough to give the results desired. The maximum speed in cleaning reached as high as 25 to 30 sq. ft. of surface per min. and the entire cost of the work, including

labor, sand, and power, was close to 1 cent per sq. ft., so that the average reached was only about one-tenth of the maximum possible, due to lost time in charging sand, changing nozzles, building and changing scaffolds, and all the usual delays in work of this character. The interference from other classes of work under way was also a big factor in reducing the efficiency of the cleaning, and it would seem easily possible on an ordinary bridge with no other work in progress to reach an average of 6 to 8 sq. ft. per min. In any event, it proved to be the one best way properly and thoroughly to clean an old bridge. Improvements in the apparatus were made by reaming out a tapering entry for the sand in the inner ends of the loose nozzle tips, and by using bent nozzles for reaching surfaces that were difficult for the operator to get at, and for the inside of latticed members.

The carburetor on the machine should also be improved so as not to clog up with damp sand, as it was nearly impossible to keep the sand dry in only damp and foggy weather, while in wet weather work soon had to be practically suspended.

The great variety of conditions on various parts of the structure required the use of twelve different kinds of coatings, each one for a specific purpose, as will be briefly described. They were red lead, white lead and zinc, Tockolith, graphite, Galvanum, cement paint, fireproof paint, turpentine asphaltum, Hermastic enamel, enamel primer, carbolineum, and cement gun coating.

The red lead was 60% ready mixed, with asbestine and other suspension pigments. This was used for the shop assembling and first coat, but for the field 3 lb. of lamp black per bbl. was added to insure further the keeping of the red lead in suspension, to increase the spreading, and to darken it so the silver-gray lead paint would cover more easily. It was intended to use this for all metal fully exposed by the sand-blast as a first or spotting coat, but a study of conditions caused a change to Tockolith, as stated.

The white lead silver-gray color paint was for the finish coat. It was 60% white lead, 20% zinc, and 20% barytes and other pigments. The white lead and the zinc were each increased from 5 to 10% in the field, so as thoroughly to cover the darker paints.

The Tockolith was substituted for the red lead to cover rust spots after cleaning, as the writer's experience for several years in painting badly rusted metals has shown that this paint absorbs the oxygen from

rust and stops further rusting. The cutting apart of girders for reinforcing disclosed that where the surfaces had been painted with red lead between cover-plates and angles before shop assembling, all the paint had been practically burned out during riveting. Therefore, Tockolith was used for much of the assembling paint in the field.

The use of Superior graphite was deemed necessary on the bottom of the copper-steel floor to resist drippings from salt water, acids, and gases. One coat was used for this purpose and the second coat was the white lead finish paint. Graphite is undoubtedly one of the few paints that should be used for assembling, so as not to be burnt out in rivet driving. Possibly better results would be had by substituting something, at least in part, for the linseed oil.

The Galvanum was used for painting a few hundred square feet of galvanized corrugated-steel roofing between the girders at the Toll Offices. Ordinary paint could have been used by first washing the galvanizing with a weak muriatic acid solution, or by adding cut shellac to ordinary paint.

The cast-steel bases and boxed-in bases of the viaduct columns were filled with water-proofed concrete, formed with the addition of Medusa compound. To exclude the water further and to match up with the silver-gray coat on the bridge, Acme cement paint was used for painting the exposed surfaces of this concrete.

The use of Acme fireproof paint for the timber floor was investigated by painting some boxes and attempting to burn them. The results were so satisfactory that it has been recommended that the cracks in the roadway and sidewalk floors be sprayed with this paint as soon as the lumber dries out in the spring, in order to prevent fires starting from cigarette and cigar butts.

The elbows and inclined sections of the 4-in. steel-pipe downspouts were painted with turpentine asphaltum as a partial protection from salt water, since it was not possible under the conditions existing to use Hermastic enamel in this location.

The Hermastic enamel was used for coating the copper-steel drainage floor as a protection against the rusting or corrosion of the steel of the railway floor system due to salt water and other drippings. The copper-steel plates were also coated with this where they had a bearing on the maple risers, and in addition the maple was given a coat. The metal was first cleaned, then coated with the enamel primer, and the

Hermastic enamel applied with special daubers, while the enamel was very hot. Even then it would only spread a distance of 12 to 18 in. Owing principally to the thickness of the plates and the constant breeze or wind down the Gorge, about 50% excess of the enamel was used and the average thickness was more than ¼ in.

Carbolineum was used for all timber treating, and it was generally applied hot with a brush, but the blocks were dipped in the kettles where it was heated. From one-fourth to one-half of coal oil was added to increase the spreading and penetration of the carbolineum. The timbers treated were the spiking strips for the highway floor, the maple risers and blocks for the tie-risers, as well as the stop-waters and other minor timber details.

The coating with cement mortar by the cement gun has been spoken of under the discussion of the skewbacks and only mention of it need be made here. The concrete abutments and walls were simply painted with a cement mortar cream.

Steel Repair Plant.

The steel construction plant used by the Terry and Tench Company, which did the construction work, was exceptionally complete. The three compressors of about 230 cu. ft. per min. capacity of free air, were of the motor-driven type such as has been used on elevated railway work in New York City. The motors are 50 h. p., 250 volts, and the whole equipment for both motor and compressor is mounted on steel wagon trucks. The electric current was obtained from the feed lines of the electric railways on either shore, and so no shut-down of the work was necessary when power was off at one end. They all fed into the same 2½-in. pipe extending the length of the bridge, and connecting to two air tank reservoirs. Frequent branches and valves were provided for connecting the hose for air drills, riveters, sand-blast machines, and cement gun. The long gun riveters were of the usual type, and the air drills and reamers were of the high-speed type running at 700 rev. per min., with drills and reamers to correspond, so that no difficulty was found in keeping work abreast. There were twelve riveters in use, and fifteen air drills and reamers.

The sand-blast machines were made by the Mott Sand Blast Company, of Brooklyn, N. Y., and as stated were three in number; the No. 1 size consuming 94 ft. of free air per min., and holding 500 lb. of sand;

while the No. 2 size consumed 94 ft. of free air per min. and held 1 000 lb. of sand. They consisted of a sand tank filled through a sieve at the top, and discharging through a conical bottom to the carburetor where the compressed air is introduced, all regulated by suitable valves. The discharge hose is of special heavy reinforced rubber, and carries a nozzle which has small hardened cast replaceable tips. These tips were those reamed out conically to allow freer discharge, and special bent nozzles were made to facilitate work which was often somewhat out of reach and partly inaccessible for the ordinary nozzle.

The oxy-acetylene torches for cutting were of the ordinary type, the burners or tips having three holes one above the other. They were used for cutting off old column bases, curving out gussets, cutting up old flange plates, and many other cuts which if made by hand would have taken hours instead of minutes. For cutting off rivet heads, the tips used for ordinary burning were slow and a special tip was used with three holes in line, which cut off an average of about 80 heads per hour and did not weld the shank of the rivets into the holes. No more valuable tool can be found for field repair work, and it can be used for any work where the danger of overheating the steel does not need to be considered.

The material was unloaded by a small "jiniwink" or shear-leg type of boom derrick, operated by an engine using compressed air, and was distributed over the highway deck as far as possible by means of a hook buggy and by several large lumber-yard buggies. Owing to the great height of the work above the Rapids and the necessity for keeping up traffic, double hitches and double slings were used, so that with constant watchfulness not one accident from this source occurred.

Rock Cliffs.

The cliffs at each end of the bridge were found to be quite solid when the arch was built to replace the Suspension Bridge in 1896, and therefore they were left with an overhang above a vertical cliff wall of 50 or 60 ft. in height. Some rock falls had occurred during the years following, one of which did some damage to the end shoe strut, really a tie, on the Canadian end of the arch. There had been some bad falls up and down the Gorge from the bridge, and as the cliffs were badly weathered, it was decided to remove all the loose rock and overhang which might, in falling, damage the arch span. This removal con-

sisted in taking off a 12-ft. overhang under the Canadian 115-ft. span, and an overhanging point directly adjacent to and north of this. The 8-ft. overhang under the 115-ft. span on the American end was also removed. The yardage was not a large total, but the rock had to be broken up into less than one-man stone before casting it over at the Canadian end, and before hoisting at the American end, on account of the electric railway down in the Gorge. The skewbacks and tracks were well protected by plank aprons. The rock was drilled in rows of holes for one cut, using hand air (or plug) drills, and the face finally left was practically smooth and vertical, but all the great superimposed weight was taken off and no future falls are expected.

Should future conditions warrant further protection, it is proposed to construct just above the skewbacks on either side a rock-catching abutment which will catch and retain a large quantity of small rock before a cleaning out would be necessary, and in the case of a large fall, would break its force and prevent its bouncing on to the steelwork. Such a device might be found useful in many similar locations where protection from rock falls is necessary.

Masonry Repairs.

The repair of existing masonry is usually more troublesome than building new; especially is it difficult to decide on a sure cure for trouble. This is usually due to the impossibility of knowing the condition of the interior of walls or masonry masses, except as its solidity may be learned from borings, and this method of testing is often far from satisfactory. Added to this difficulty, there was at Niagara the old suspension bridge abutments and anchorages in use for the abutments of the railway viaducts. These were built of limestone in 1855, making them 64 years old, and there were no plans in the files from which to determine their thickness and the character of construction.

Fortunately, the writer found at Van Nostrand's one of the original Roebling monographs on the Suspension Bridge, published in England, and in this were plates giving the masonry details. The front walls of the abutments which carried the girder spans were found to be about 9 ft. thick at the ground line; and this was later checked up when weep-holes were bored through them with a Burley drill carrying a 5-in. drill bit, in which to insert 4-in. steel drain pipes that

were also to extend through the new concrete face wall, which is described later.

The old suspension bridge towers, as has already been mentioned, were replaced with steel by Mr. L. L. Buck, and when the stonework was torn out from between the new steel members of the towers, the rock was found to be completely shattered or broken up in the interior. The abutment wall under the girders at the Canadian end was quite shattered from frost and traffic combined, so that it worked under passing trains and was considered to be unsafe. A heavy timber bent was erected to carry the traffic, and the face of the wall barred out to a depth of more than 1 ft., or to where it was quite compact under pressure. Forms were then built of 2-in. surfaced lumber and a new concrete face wall or carrying abutment was constructed. The girders were carried on this abutment using new cast-steel shoes, with new stiffeners riveted on the girders above the new support and on the webs between this and the first old pair, to care properly for the shear. The old end of the girder had the bearing-plates removed and now carries the track as a cantilever for the distance from the new bearing to the old back-wall.

The stone pedestals removed from beneath the viaduct columns on account of being cracked were found to be badly shattered, much the same as noted for the old towers and the old abutments. This led to devising a thorough method of protection of the skewback masonry of the arch from weathering, as it had only begun to show the effects of 22 years of exposure to the rigorous climate. The original scheme adopted had been simply to coat the limestone courses below the granite copings with a cement-gun finish, but on the developments in regard to the other masonry, it was decided to armor all four of the skewbacks as shown in Fig. 23, which also shows the thrust wall between them, which is described later.

The armor was shot on with a cement gun, the proportions of the coating being one of cement to one and one-half of clean sharp sand, with an air pressure of 40 lb. This coating was about 6 in. thick at the top, and more than 12 in. at the bottom of each skewback, as was necessary to get a regular batter line. The coating was reinforced by a triangular wire mesh which had first been fastened closely over the old stonework by spikes driven into the cement joints. This mesh had No. 9 longitudinal wires 4 in. apart and No. 12½ cross-wires 8 in. apart.

Just below the old coping a new coping was shot on, reinforced by a bowed-out strip of wire netting. The old coping therefore appears as a pedestal block.

The stone skewbacks were quite high and slender, especially on the American side, and under the most severe assumptions as to stresses from wind on the unloaded bridge, which is exceptionally high and exposed, it was found desirable to carry the directly applied stresses from the new sway bracing down to the solid rock foundation strata. The drilling of any holes through the skewback masonry for efficient tie-rods seemed undesirable, and, therefore, the skewbacks were connected by the thrust-walls shown in Fig. 23. These walls were 4 ft. thick, with a plain coping, and arched at the bottom to allow falling rocks and débris to pass through or be cleaned out, so that they would not eventually become retaining walls. Footings were built next to the skewbacks to carry each end of a wall. The reinforcing consisted of sixteen 1-in. deformed bars in diagonal lines from the top of one skewback to the bottom of the other, each bar being placed 6 in. into the masonry at each end in holes drilled with the plug drills. These holes were put into the tops of the high American skewbacks, slanting downward, so as to afford some tension anchorage. Each set of sixteen rods was hooped into a column as shown. The arched bottom was reinforced to act in conjunction with the shear rods, simply to prevent cracks in the wall.

The thrust-walls and the armor coating not only serve perfectly the structural purposes for which they were intended, but as may be seen from Fig. 23, they add much to the appearance of the structure, especially as to solidity and stability. They are also designed to serve as part of the enlarged skewbacks that will eventually be needed to carry additional ribs for a four-track bridge.

Deductions.

The data given as to increase of locomotive and train loads, supplemented by such other like data as have been published and which can easily be obtained, should be made the basis for negotiations by the Society with the leading railway societies and with locomotive and car builders as to the allowable increases for a reasonable time in the future. This would make it possible to plan more definitely than could otherwise be done for the strength of future bridge construction. The great

requirements for steel in all classes of work in which iron and steel are utilized makes it certain that from every viewpoint raw supplies for the manufacture of iron and steel structures must be conserved.

The information presented as to the great diversity of possible bridge loadings now in use, and the distribution of stresses occurring in structures as compared to calculated stresses, indicates the great futility of over-refinement. Although the system of composite loading proposed may prove to be no more than tentative, it is certainly to be hoped that it will be made the basis by the Society, and others interested for the adoption of a comprehensive, logical, and simple system. This can certainly be done, and be a credit to that branch of engineering which lays claim to being the most scientific of the many specialties now comprised under the general head of Civil Engineering.

The strain-gauge results obtained, while not perfect, certainly indicate the necessity of making both dead load and live load readings of representative members of all important structures. The gauge points should be permanent, put on the members and read before erection, to the end that the bridge engineer may at any time easily make a check of actual stresses and their distribution, as caused by standard trains. Should such a scheme be instituted by the large railway systems, it would then be possible, with the aid of the United States Bureau of Standards, to obtain extensive and conclusive data as to impact on bridges, as to secondary stresses, as to distribution of stress over the cross-section of members, and, finally, to obtain much valuable data as to the strength of compression members and members undergoing reversal of stress. The needed information to supplement this kind of data on compression members could also be easily obtained if each railway would substitute for full-sized eye-bar tests, the testing of one or two compression members for each important structure. This would probably corroborate recent data from tests that indicate the need of better compression member details, rather than any change in the sections themselves or in their make-up.

The stress readjustment carried out in the Niagara Arch, by using toggles to change the center shim, is seldom possible of attainment in the ordinary structure, but it is not too much to say that there is a logical way possible for the revision of most large structures, and the strain gauge would play no small part in accomplishing the desired result. The great lesson to be learned from this Niagara revision and

the examples of the first Quebec and the Blackwell's Island Bridges, together with the success obtained for the Hell Gate Arch, is that the real design of a structure does not lie entirely in computing stresses and designing details, but in the greater problem of the most fitting general design for a given location and as a general scientific problem. The writer does not hesitate to repeat a statement often made heretofore, that engineers cannot properly design a bridge unless they have made every effort to give it the proper æsthetic consideration, with the assurance that in so doing they need not violate the principles of scientific design, nor of the truest economy.

The investigation at Niagara emphasizes the need for the engineer of any important structure to leave the most complete data possible in the files of the owners. This should not merely include blue prints and specifications, but copies of all computations, stress sheets, and data as to the construction, and of practical details of the work. Such information is not only of value as relating to the particular bridge, but will result in more exact scientific data being obtained whenever such a bridge is under investigation.

The scientific investigation of the Niagara Arch was made financially possible through the courtesy of George L. Burrows, Assoc. Am. Soc. C. E., President of the Niagara Falls International Bridge Company, and Charles Riordon, President of the Niagara Falls Suspension Bridge Company, joint owners of the bridge. The field and office work was carried out with the assistance, during 1918 and 1919, of C. T. Morris and A. H. Fuller, Members, Am. Soc. C. E. The Resident Engineer on construction was H. A. Reid, Assoc. M. Am. Soc. C. E.; and acknowledgment is also made for the assistance rendered by F. L. C. Bond, Chief Engineer, and H. B. Stuart, Structural Engineer, of the Grand Trunk Railway, and by P. L. Wolfel, M. Am. Soc. C. E.; also to H. G. Dickinson, Superintendent for the Bridge Companies; and to The Terry and Tench Company and the Lackawanna Bridge Company, contractors for construction and fabrication, respectively.

DISCUSSION

Mr. Ammann.

O. H. AMMANN,* M. AM. SOC. C. E.—The author deserves much credit for the careful and thorough manner in which the revision of the Niagara Bridge has been carried out, as well as for the elaborate experimental investigation conducted in connection therewith. The redesigning and reinforcing of the bridge is in accordance with the most advanced practice and should give the structure an additional useful life equal to that of a new one.

Experimental investigations such as those made on this bridge are most valuable in giving engineers knowledge of the exact stress conditions, and for this purpose cannot be too strongly encouraged nor too painstakingly carried out. The knowledge gained, however, must be applied in a broad sense and above all with a view to the improvement of the excellence of the design rather than to economy by excessive refinement.

Bridge building is still an art, and as such its success depends largely on broad vision and judgment in the combination of the many intricate and changeable elements which enter into the design and which cannot, by themselves, and still less in their combined action, be determined exactly by scientific laws. A liberal margin in the selection of these elements therefore is justified. From this point of view, the speaker takes exception to some of the suggestions for revision in bridge design made by the author.

The author proposes the abandonment of Cooper's engine loading and the substitution therefor of the composite moment and shear curves or tables derived from some of the recent heaviest locomotives.

Although Cooper's loading is not an ideal one for all cases, it covers, probably more than any other loading proposed, the widest variety of engine and train loads. It has become a standard with which other loadings, past and present, are comparable in weight and axle loads, as well as moments and shears produced. Compromise moment and shear curves do not allow as simple and tangible comparison, because they do not represent any particular loading; and what guarantee is there that such curves may not, within a short time, become even less representative of actual loads than Cooper's loading? They at least are not representative of the large number of engine loads which are still prevalent on many roads and which must be taken into consideration in establishing a general representative loading.

Even if only the four extreme actual loadings selected by the author are taken as a basis, the speaker would consider Cooper's E-66 fairly representative, because the greatest excess of moments and shears from these extreme loads over those from E-66 would not be more than 15% and this only in very short spans. This is not excessive, con-

* South Amboy, N. J.

Mr. Ammann.

sidering that the moments and shears from the four locomotives differ as much as 40% among themselves.

The author's composite moments are in excess of those from the actual loads by amounts up to 30%, not considering the additional 25% allowed for future increase of the loads; and inasmuch as this excess would affect the great number of small spans, it would not seem to be conducive to good economy.

For spans greater than about 250 ft., Cooper's loading appears to give excessive results. It must be considered, however, that the excess is not due so much to the engine loads as it is to the train load behind the engines. This load is more apt to increase in the future than the average weight per foot of the engine. Moreover, in longer spans, an excess in the live load produces a proportionately much smaller excess in the total stresses or sections than in smaller spans.

The speaker, therefore, believes that there is not sufficient ground for the abandonment of Cooper's loading, but that, on the contrary, it will become, even to a wider extent than in the past, the standard loading for designing railroad bridges.

The author introduces a new impact formula based on the train speed. Although in isolated cases it may be justified to design a bridge for limited speed, as was done in the case of the Niagara Arch, it is questionable whether a formula can be established which would be generally applicable, because impact is dependent on many factors other than speed.

In the Niagara Bridge the impact for various speeds was determined from careful readings, and there can be no doubt that the curve thus established is correct and justified a reduction in the impact allowance in redesigning the bridge. For other bridges the curve may be entirely different, mainly because the critical speed causing maximum impact is dependent on the type of bridge, span length, dead weight, and even on the live load itself. Only extensive series of tests would shed further light on this question, and for lack of additional data it is safer to design bridges with full impact allowance.

The author's adoption of high permissible unit stresses of about two-thirds of the elastic limit and his application of them to the combined stresses from all sources, is a step in the right direction. In the speaker's opinion, it would not have been necessary, however, to include the secondary stresses in the maximum stresses, because they are nowhere excessive except in a few members, which, for constructive reasons, have an excess section.

In general, secondary stresses may be assumed to be safely covered by the one-third margin of safety below the elastic limit, provided care has been taken in the design to avoid large secondary stresses. It is safe to say that, by reason of greater rigidity and superior details,

Mr. Ammann. bridges designed according to good modern practice and in which excessive secondary stresses are avoided, have a greater margin of safety than former bridges in which the secondary stresses were not even investigated.

The author's compression formula is well chosen in view of the experimental data gained during the past few years and, as a simple straight line formula, deserves general adaption in cases where high unit stresses are chosen.

Recent attempts to introduce apparently more refined formulas are not warranted. What engineers need more than an ideal formula is improved practice in designing compression members, so that they will develop the full strength attributed to them.

Mr. Bentham. C. BENTHAM,* ESQ.—The author states that the bridge was constructed of what is termed in the United States as medium steel, and that the rivets are of soft steel; he also mentioned that some of the rivets came loose. This may be due to a number of causes. In bridge construction in England, especially over tidal waters, trouble has been experienced with the rivets in steel bridges, and it has been thought that electrolytic action between the rivets and plates when of different material is intensified by the salt water. In the Forth Bridge, numerous rivets have been replaced because corrosion has occurred between the plate and the head of the rivet. Corrosion of the plates appears to be more intense with those of modern steel than with those of wrought iron. The bottom parts of the tubular supports of the Forth Bridge are rusting away because of waves beating over them.

In answer to the question as to whether the renewal of the girders was on both the railroad decking and the road decking, the author states that only the girders on the railway deck were renewed as the salt water from the refrigerator cars did not get down to the lower deck. One of the photographs shows the bridge with all the track removed and all the girders exposed, and yet the author states that the work continued without interference with traffic, explaining that in a double-track bridge, one track was kept clear while the other was being renewed. The floor-beams, he states, extended across the width of the bridge, and the engineers shored up and took the cover-plates off the floor-beams without interference with traffic.

This point is not quite clear to the speaker because it looks, from the photograph, as if the whole deck had been taken off. In regard to this loading weight, it has been found that many of the earlier English bridges are far too weak, according to the present-day formulas. A number of them have been stiffened, but there is one bridge in particular, constructed by Brunel at Chepstow, about 1840, where it was found that the stress was above the elastic limit of the wrought iron

* Manchester, England.

used in the construction. That bridge stood for forty years. It has been stiffened, because the weights have been increased; with the loads as calculated, however, it should have broken down many times in those years, but it did not do so. As the weight of the locomotives increased, the speed was reduced to a limit of 10 miles per hour over the bridge. Presumably, this is partly what was done at Niagara before the bridge was stiffened, the speed of the trains being gradually decreased as the train loads were increased. Mr. Bentham.

The critical speed of the average train in relation to any particular bridge is an important factor, because, when ascertained, the members of the bridge may be safely stressed more nearly to their elastic limit if this particular speed can be avoided.

The author has mentioned the greater utilization of the water power of the Rapids, and he is probably aware of what is now being done. The Ontario Hydro-Electric Commissioners are at the present time carrying out a scheme which utilizes the entire head of the water with the exception of that which is necessary for hydraulic flow. The water is being used up to a head of about 300 ft., whereas at the existing power stations the head is about 160 ft. The benefit of the Rapids below the Falls in addition to the Rapids above the Falls, is being obtained.

J. A. L. Waddell,* M. Am. Soc. C. E. (by letter).—This paper is one of the most interesting and instructive treatises on bridgework that has appeared in a long time. The labor involved in its preparation must have been very great, and the author deserves the hearty thanks of the Engineering Profession in general, and of bridge specialists in particular, for his able efforts on behalf of his brother engineers. The historical features of the paper are specially interesting and valuable, because far too little is either recorded or generally known concerning the history of engineering. Again, the dissertation on live load augmentation is most instructive. The writer agrees with Mr. Fowler that the time for abandoning the Cooper type of loading has arrived, but thinks that curves of equivalent uniform live loads should be given in specifications for design, rather than the irregular concentrated loads. These curves should not follow exactly the equivalents for any particular locomotives, but should envelop on the diagram most of the plotted points for the equivalent uniform loads of a number of modern or possible future engines. In predictions concerning future live loads, it must not be forgotten that for the roadbed the present style of track is loaded about all it will stand, so that any seriously greater burden will have to be distributed in some more uniform manner than is attainable by the arrangement of rails and ties used generally in America. Mr. Waddell.

* New York City.

Mr. Waddell.

In pointing out "the need of better compression member details, rather than any change in the sections themselves or in their make-up", Mr. Fowler has struck the keynote in reference to the proportioning of struts, concerning which there has been so much discussion of late; and it is evident that, like the writer, he does not see any real *raison d'être* for the panic about the alleged weakness of the old standard compression formulas initiated by the Society's Special Committee on Steel Columns and Struts in its final report.*

The writer is in accord with the author also when the latter says that "the real design of a structure does not lie entirely in computing stresses and designing details, but in the greater problem of the most fitting general design for a given location", and also when he states that a proper consideration of æsthetics will not violate the principles of scientific design nor those of the truest economy.

Bridge designers in general, and especially young men who are studying the specialty, will do well to hearken to these words of wisdom from one who, by years of experience in designing and construction, deep study of theory and principles, and the writing of standard works, has established himself as one of the highest authorities in bridgework.

Mr. Belzner.

THEODORE BELZNER,† ASSOC. AM. SOC. C. E. (by letter).—The author is to be congratulated in presenting to the Engineering Profession, and to the members of this Society in particular, such an interesting and important paper.

In reference to the comparative readings to determine the respective accuracy of the Berry, Howard, and Fuller-West strain gauges, it would be of interest to the writer to know the relative accuracy of the Howard gauge for static load readings, as compared with the other two instruments.

During the strengthening of the end spans of the Williamsburg Bridge some years ago, the Department of Bridges of the City of New York, in co-operation with the United States Bureau of Standards, under the auspices of James E. Howard, Engineer Physicist, conducted a comprehensive series of extensometer investigations on the important truss members at the main towers of that bridge and on the legs of the Brooklyn intermediate towers.

The writer had the good fortune of being assigned to these investigations by the late Austin Lord Bowman, M. Am. Soc. C. E., then Chief Engineer of the Department of Bridges. He also had the pleasure of being associated with Mr. Howard from the beginning to the completion of this work, and became thoroughly grounded in the importance of precision in such work in order to secure the best results.

* See p. 1583.

† Brooklyn, N. Y.

Mr. Belzner.

A 20-in. Howard extensometer was used throughout the entire investigation, and the precision attainable with the strain gauge and its reliability in measuring stresses proved it to be an invaluable aid to the Bureau of Design of the Bridge Department in arriving at the conclusions on which the successful issue of the work was based. Its merits were illustrated in certain vital operations, especially in connection with the various stages of the wedging operation, and in the transferring of stresses from the old to the new diagonal members of the end trusses. The accuracy of these results has been discussed by Isidore Delson,* Assoc. M. Am. C. E., Assistant Engineer, Department of Bridges.

D. B. Steinman, M. Am. Soc. C. E., in his exceedingly valuable paper entitled, "Stress Measurements on the Hell Gate Arch Bridge",† has described the Howard strain gauge so fully, the methods of its operation, and the precaution required in its use for precise work, that little can be added, except, possibly, a few references to methods of making the measurements. After the preliminary work on the members to be measured, the most important point to be considered is in obtaining consistent measurements, and this point cannot be emphasized too strongly. In order to secure such results, which are absolutely essential, persistent patience and painstaking thoroughness, combined with good judgment on the part of the observer, are required until he gains perfect control of the instrument, after which he will experience little or no difficulty; and it is upon the observer that the value of such measurements will depend.

Another fact to be remembered is that all measurements should be taken during the early morning hours, preferably before sunrise in summer, and on cloudy days, in order to eliminate any possible internal temperature strains in the members.

The writer's experience with the operation of the Howard extensometer, extending over a long period of measurements, has been that the use of the inside milled head is preferable, on account of the sensitiveness of touch for contact, and he agrees with Dr. Steinman‡ that it "is probably the simplest and most accurate field instrument for measuring strains."

It may be of interest to state that, in September, 1913, during the stress investigations on the Williamsburg Bridge, Mr. Howard tried out, on the stiffening trusses, a new strain gauge which he had designed for measuring live load stresses. This instrument is about 3 in. long and 1¼ in. wide, and might be properly termed a "scissors-gauge". This gauge has been fully described in an article entitled "A New Strain Gauge for Live Load Stresses".§

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1087.
† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1040.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1051.
§ *Engineering News*, October 9th, 1913, p. 701.

Mr. Schmitt.

F. E. Schmitt,* M. Am. Soc. C. E. (by letter).—As the first instance of large-scale bridge reinforcement on the basis of thorough analysis by measurement of stresses, the work described by Mr. Fowler is not only notable in itself, but also presents a valuable model for future work in this field. The revision was greatly simplified by the excess of strength originally provided in the arch rib, which made it unnecessary to add material to the arch members. Nevertheless, the impressive fact is made known by the author's account that the operations described extend the life of the Niagara Railway Arch indefinitely, without impairment of efficiency in the utilization of its material. This is a new achievement, and one of great importance. It is particularly noteworthy because it is based on investigations which are truly scientific, including separate studies of condition and behavior of structure, nature of service, and capacity of material. Since these studies were essential to the results, it is fair to assume that similar investigations, at least for service stresses, will be found necessary in future cases of reinforcement of steel structures, if the results are to measure up to the possibilities.

The reinforcement of large bridges has been undertaken but rarely. In large part this must be charged to the fact that bridge practice progressed continuously, and that when a bridge was outgrown by the traffic loads it was almost necessarily replaced by a new one of better type. In the railway bridge field, conditions have changed sufficiently to eliminate this necessity, so far as structures of recent date are concerned. As bridges are built to-day, they may be considered virtually proof against "wearing out" or obsolescence of like character. Change in amount and kind of loading is the chief future contingency, and this may be dealt with by reinforcement. Therefore, less bridge replacement and more reinforcement work would appear to be in order for the future.

Bearing directly on this question is the remarkable demonstration of the longevity of well-built and well-maintained steel bridges afforded by the Niagara Arch. After having done service for more than twenty years, the bridge proves to have been essentially free from deterioration in its primary parts. Steel bridges have been regarded as inherently short-lived, but the Niagara example makes this view untenable. It shows that steel bridges may be made permanent structures, the original investment in which is a fixed asset. To realize permanence it is necessary, however, that the designer exercise sufficient foresight to guard against destruction of service value by obsolescence, that maintenance be attended to, and that a high degree of thought and skill be applied to reinforcement (when this is called for), perhaps

* New York City.

even greater skill than is ordinarily applied to bridge construction and replacement. Mr. Schmitt.

In connection with the first of these requirements, it is pertinent to remark that, since designing for indefinite future increase of loading is usually impossible, attention at the time of original design to the likelihood of future reinforcement should be worth while. It is safe to say that few if any bridges have been planned with this point in mind, but there is no reason why the design should not be worked out so as to simplify reinforcement at a later period. An incidental advantage would be gained, which is by no means negligible: When the planning of a bridge is guided by the purpose of producing a permanent structure and providing in advance for future operations to adapt it to changed service, a design of far higher character will result than when the controlling thought is that the structure will have only a moderate life, on account of deterioration or change of service requirements.

To make reinforcement ideally efficient, both old and new materials must be utilized to full capacity. Something has been done in developing methods to secure this object. The remarkably well conceived crown-shim readjustment to distribute the dead load stresses of the arch more favorably, and the toggle device used in reinforcing the posts of the Niagara truss spans, as well as the successful use at Poughkeepsie of heating in attaching reinforcing plates, fall in this category, and deserve high appreciation on the score of simplicity and effectiveness. However, there can hardly be a doubt that methods to accomplish equal division of service between old and new material will be developed much more extensively in the future than in the past. In work so variable and so dependent on the engineer's resourcefulness as bridge reinforcement, it seems to be inevitable that a much broader range of available methods must be worked out to meet the many complex sets of conditions arising in practice. It may be expected therefore that with the expansion of reinforcement work the development of the structural art will enter on a new phase.

As the first step in reinforcement work, or even in the preliminaries leading up to a decision on the question of replacement or reinforcement, is the determination of actual effects of service, the relation between calculation and measurement of stresses calls for consideration. In the writer's opinion there is usually enough uncertainty about the distribution of stress in a structure, particularly in one that has been long in service, to make strain-gauge measurements indispensable to a satisfactorily thorough knowledge of service effects. Relatively few comparisons of calculated and measured stresses have been made, from

Mr. Schmitt.

which a good judgment of the relation between them could be formed; but these indicate that even with respect to primary stresses the check is only approximate, especially under moving loads, while stress variations over the cross-sections of members, and numerous other auxiliary effects, create very serious divergences between measurement and calculation. In considering questions of reinforcement, the measured stresses will naturally be accorded determinative weight, and calculation will be regarded only as a check. It is thus possible by means of measurement to adapt the distribution of material in a structure much more closely to the actual stresses developed in service than is possible in constructing a new bridge, a point worth special consideration as suggesting that a distinctly higher efficiency can be reached in reconstructing an existing bridge than in building a new one.

With regard to the use of strain gauges for the measurement of live load stresses, the fact is not very prominently dwelt on by the author that even in a constrained structure, such as the Niagara Arch, these stresses are as definitely calculable as in simple structures, except for two factors: The influence of details and possible looseness at a support. The latter can be determined and evaluated by inspection; the influence of details, on the other hand, is likely to be difficult to estimate, because of the lack of suitable data. Deflection observations on bridges of either simple or constrained type would supply such data, but observations of this kind are rare.

Apart from the question of influence of details, the primary live load stresses in the Niagara Arch should have been determinable as accurately by computation as in any type of bridge. The results of the measurements and comparisons make it clear, however, that secondary and impact effects are of such magnitude as to give them an importance from the designer's viewpoint equal to that of the primary stresses; and very obviously, according to the results, these secondary and impact effects must be measured by the strain gauge rather than be approximated by rule-of-thumb formulas which were framed to provide allowances suitable in new design.

Dead load stresses appear not to have been a matter of much concern in the Niagara case. It is to be considered, however, that where such service is placed on metal as in the revised Niagara Arch, in other words, where a high efficiency of utilization of material is aimed at, the dead load stresses should be known quite accurately. The crown jacking which was done to give a datum point for the calculation of dead load stresses supplied no further information than that it removed the statical uncertainty due to constraint. It did not give fuller knowledge of actual dead load stresses than is available in the case of a simple structure without any measurement whatever. It is of some interest to speculate on what methods might have been used, and at

Mr. Schmitt

what cost, to make direct measurements of the dead load stresses existing in one or two important members. The writer, however, believes the case is most valuable in again directing attention to the proposal made some years ago by Mr. James E. Howard, that gauge points for measuring stress deformations be provided on the members of important structures before erection. When such points are placed, in permanent fashion, and initial measurements are made, stresses can be measured in a reliable manner at any time during the life of the structure. The measurements made by Mr. Howard on the Armour-Swift-Burlington Bridge, at Kansas City, and on one or two other bridges, besides several buildings, constitute a thoroughly practical demonstration of what may be done in this direction. The importance of his suggestion and its practical demonstration do not seem to have been fully appreciated. Renewed reference to Mr. Howard's work is believed to be justified by the fact that, in so important an undertaking as the Niagara revision, a distinct uncertainty surrounds the dead load stresses which constitute a very respectable proportion of the total stresses and hence of the utilized strength of the members.

With regard to the whole subject of the permanence of bridges, the facts brought out in the paper as to the condition of the floor-beam flanges and the stringers are highly significant. The figures given as to deterioration of the floor are in alarming contrast to the statements as to the excellent condition of the primary members of the bridge. Although it is true that floor material usually can be renewed without great difficulty, yet the rapid loss of strength due to corrosion would make it seem a desirable safeguard and economy to attempt to provide better protection for floors than is given by the ordinary undrained, open-tie, floor construction which is virtually standard in railway bridge practice. The Niagara drip-plate protection is an ingenious and praiseworthy solution of the problem encountered there, but its value for general use is likely to depend on the maintenance which it requires. Experience with these drainage plates will need to be awaited as to this point. Although the last report of field tests of the Committee on Corrosion of the American Society for Testing Materials, made public after the presentation of the author's paper, indicates that copper-bearing steel is distinctly more resistant to corrosion than non-copper-bearing steel, yet the differences are not large enough to make the plates give promise of anything beyond a very short life. However, in spite of the use of thin steel plates in a substantially horizontal position, and wooden packing blocks which are also likely to be subject to rapid deterioration, the arrangement adopted has the great advantage of protecting thoroughly all the carrying parts of the structure and concentrating the deterioration in easily renewable parts, where ordinary maintenance can take care of them and continue the protection of the structure unimpaired.

Mr. Schmitt.

Besides bringing to the front important matters concerning bridge reconstruction, however, the paper raises numerous open questions and doubtful points in bridge design. Among these are railway loadings, impact effects, permissible unit stresses, the importance of secondary stresses, the suppression of vibration, etc. The vital way in which these points were involved in the Niagara work seems likely to open up anew a discussion on various phases of bridge construction, which by many have been regarded as practically settled. It is hoped, therefore, that full discussion of the matters in question will develop, and that the experience and opinions of practical bridge engineers may be brought out in a representative way. Although the paper gives only a small range of data on some of the matters concerned and most of these data are of quite inconclusive character, the method of attack used was effective and satisfactory, and (since sound and serviceable results were obtained) the data given in so far as they bear on bridge design are entitled to careful consideration.

The wide departure of the procedure followed in the Niagara work from that prescribed in standard bridge specifications suggests that a committee of this Society on bridge specifications, as recently proposed, would find ample work to do. One point with which such a committee might have to concern itself is unfortunately passed over in the paper without discussion, namely, the treatment of reversed stresses. In consequence, it is not very easy to determine the relative margins of safety in the main chord members and the web of the structure. According to the prevailing design practice of to-day, members subject to stress reversal are proportioned with increased section and still further increased connections, but there appears to be no agreement as to whether this is done because stress reversal has an injurious effect on the metal or its connections, or whether it is a contingency allowance for change in loading (in view of the fact that increased loading has most unfavorable effect on these particular members). The writer knows of no data from test or service observations to support the view that stress reversal, such as occurs in bridges, has any effect on the metal aside from that measured by the maximum stress. If reversed stress clauses are really intended to provide for load increase, this purpose might be expressed more properly in the wording of the clause. In any event, reversed stresses are not likely to have the same meaning in reconstruction as in new design.

Two points bearing on design requirements will be commented on here, namely, reduction of vibration and the selection of a loading system for design.

So far as the paper gives information, it is not possible to judge whether the reported virtual elimination of sway and vibration by putting in rigid bracing between the arch posts was due to the increased section of bracing thereby provided, or to the fact that the new brac-

ing is of the rigid type. Conceding that prejudice against light rod bracing is well founded, one may fairly ask if any data exist to show whether rod bracing of more ample proportion, well detailed, and properly adjusted, may not make a structure fully as rigid as stiff bracing. On this matter of rigidity, existing specifications fail to give any useful advice. Some guidance seems desirable, however, for there is a wide interval between the extremely light sway rods of former days and the surprisingly massive bracing found in many modern bridges. The best practice may lie between these extremes. Further, considering rigid bracing, it is proper to take note of the lateral secondary stresses which it induces in the main members; the effect of these secondaries is beneficial in only a very minor degree, and, on the other hand, may be distinctly harmful, particularly in tension members. Mr. Schmitt.

Whether or not the adjustment of the sway rods by the strain-gauge check carried out in 1918 was of service in stiffening the bridge, is not explicitly stated by the author, but it may be inferred that some decrease of side sway was thought to have resulted. Was the author able to form any specific opinion on the degree of initial stressing and the ratio between the sway-rod section and the main section, which will tend to assure that the bracing will be taut and the structure rigid under all service conditions? Also, in dealing with the old rod sways between the long posts, did he observe anything that would tend to show whether ordinary loop ends are too yielding for efficient service of sway rods fitted with such ends?

Commenting on the author's recommendation of a composite load system, selected to correspond to the maximum of various modern traffic loads, O. H. Ammann, M. Am. Soc. C. E., dissented from the view that Cooper's system of loading should be discarded, and referred to the moment curve in Fig. 4, as demonstrating the satisfactory character of the Cooper loadings. This diagram shows clearly, however, that the old system has a marked deficiency when applied to moderate span lengths, and an even more marked excess when applied to long spans. Practical considerations would counsel an opposite relation. In any event, it is plain from the diagram that maximum present-day traffic is not represented by any one Cooper loading for all spans. To produce a bridge of uniform strength under modern traffic, it is necessary to design the floor for one Cooper loading, the chords for another, and the web members for one or more other loadings; and in the case of a multi-span bridge, different spans must be designed for different sets of Cooper loadings. Thus, the Cooper class name is no longer a designating index number of the capacity of a bridge; calling a bridge an E-55 structure does not mean that it is adequate to carry a certain class of modern traffic. This is a practical disadvantage the importance of which the bridge engineer is more apt to underrate than the bridge user.

Mr. Schmitt

A load system more in accord with present-day railway practice would avoid this disadvantage. It is not wholly clear, however, that the substitution of another system which also requires a group of loadings to represent a single weight of traffic, as proposed by the author, would be an improvement. It should be possible to discover a suitable single loading, whether of the wheel load or of the uniform-load type; such a system should have more regard for probable future development in load distribution than for present-day loading.

Before a load system is finally formulated, it might be proper to take note of what was shown by the earliest work* of the Special Committee on Stresses in Railroad Track, namely, that the weight of a group of concentrations on an ordinary roadbed is distributed to such an extent that the effect of the individual wheels virtually disappears. It may be that a similar distribution of load effect occurs when the track is supported by a bridge floor, and some of the author's statements indeed suggest this fact. In that event, the problem of representing train effect by a conventionalized loading system would be much simplified.

In regard to the behavior of bridges, it is necessary to take careful note of one surprising fact reported by the author: That after the readjustment of the dead load stress in the arch at the crown, the completion of the readjustment after the change of shims occurred very gradually. It would appear from this observation that even in a bridge of fully elastic type, inelastic elements have an unexpectedly large influence. Without knowing more about the circumstances than is reported in the paper, one can only judge that the floor stringers, and, perhaps, in some small degree, the rails, were responsible for the lag in adjustment. In any structure likely to be affected by similar inelastic lag, it would evidently be necessary to take particular precautions in conducting a deflection test, in order to obtain a true reading of elastic deflection.

Taken as a whole, the facts developed in Mr. Fowler's investigation, and throughout the subsequent revision of the structure, constitute a most gratifying testimonial to the excellence of the design embodied in the original bridge. It does not seem to be extravagant to say that the design was a generation ahead of its time.

Mr. Lang.

P. G. Lang, Jr.,† Assoc. M. Am. Soc. C. E. (by letter).—The author states that "the logical procedure for new bridges, however, seems to be to abandon the Cooper loading entirely."

The use of the Cooper loadings has been the subject of much discussion. The engines shown in Columns 1, 2, 3, and 4 of Table 1 cover moments for engines of the Mallet and Santa Fé types only. If a careful study is made of all the various types of engines in use on the railroads to-day, and these engines are compared with the

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1191.

† Baltimore, Md.

Mr. Lang.

Cooper series of loadings, the fact will become apparent that the Cooper loadings reflect the composite of all types of engines for all span lengths more nearly than any other typical loading which may be used. This matter has been very fully discussed in a paper* written by Mr. W. S. Bouton and H. T. Welty, M. Am. Soc. C. E.

In regard to the new railway floor, the statement is made that plates and angles, originally from $\frac{3}{8}$ to $\frac{1}{2}$ in. thick, were wasted away on an average of 50%, and that, in extreme cases, only a thickness of from $\frac{1}{8}$ to $\frac{1}{16}$ in. remains.

The writer's wide experience in the inspection and observation of old bridges inclines him to the belief that this statement, in the form published, conveys an impression at variance with that intended by the author. It appears to be improbable that a plate $\frac{3}{8}$ in. thick would be reduced, by corrosion, to a thickness of $\frac{1}{16}$ in. The writer's observation of such cases has practically always indicated that plates and angles corrode when placed in a horizontal plane, and that the corrosion begins at the edge. In the case of the horizontal leg of an angle, for example, the edge may be corroded to a thickness of $\frac{1}{16}$ in., but, at the root, the original thickness is obtained.

In regard to the highway floor, data concerning the axle loads and wheel spacing of the 15-ton motor truck used in the design would be a matter of much interest. For a bridge of this character, on an international highway, a 15-ton motor truck is, it would seem, not an excessive live load to use in designing the floor.

In a bridge carrying both railroad and highway traffic, it appears somewhat inconsistent to design the roadway floor for a 15-ton motor truck and the railroad structure capable of carrying Cooper's E-80 loading. That is, in the design of the highway structure, the margin of increase over actual current requirements appears small when compared with the greater allowance made in the case of the railroad bridge. It may be, however, that the highway floor has an ultimate capacity considerably in excess of a 15-ton motor truck. In this connection, it would be interesting to know the maximum load which may be passed over the highway floor.

Mr. Howard.

James E. Howard,† Esq. (by letter).—It is gratifying to have another example presented of the use of the strain gauge on so important a piece of work as the Niagara Railway Arch Bridge, conducted in so satisfactory a manner. It has seemed that the Engineering Profession at large has not adequately realized the practical advantages which the use of the strain gauge affords. In the hands of those skilled in the making of refined measurements, and applied generously to

* Appendix D, *Bulletin No. 223*, Am. Ry. Eng. Assoc.

† Washington, D. C.

Mr. Howard.

engineering works of magnitude, no errors of computation, fabrication, erection, or subsequent use could reasonably escape detection.

Reference marks established on permanent structures can be re-examined; exact information on the distribution of stresses in the primitive loading of the structures and on their subsequent endurance of dead loads can be obtained; the re-distribution of stresses after repeated live loads have been received can be ascertained; and modifications in built-up members resulting from exposure to thermal changes, if such occur, can be determined.

It is a refined method of examination. The strain gauge as an instrument is a simple one, its adaptability of wide degree. The manipulative skill requisite to obtain reliable results is not in the possession of all. Fully realizing the care which must be exercised in its use, the strain gauge which bears the name of the writer was devised with the view of minimizing personal and instrumental errors. It has so chanced, however, that certain details of design which were carefully avoided have been used in other gauges where they appear to be held in esteem, as constituting the meritorious features of the modifications.

It is believed to be desirable to use a rectilinear movement in an instrument of this kind, and this was provided in the present form of gauge designed for field use, but which is merely a modification of an extensometer designed by the writer and first used some thirty-five years ago. The prototype of the present instrument is still being used for strain-gauge purposes, and is adapted for any gauged length between 1 in. and 30 in. The usual range for a 20-in. strain gauge is $\frac{1}{2}$ in. with zero line at mid-stroke. The readings of a rectilinear instrument are reliable at all parts of its stroke, which is limited only by the capacity of the micrometer screw used, which if desired may have a range of 1 in. or more.

On measured strains in steel members, a limited range only in capacity of the gauge is essential. On other classes of strain-gauge work, $\frac{1}{2}$-in. capacity has not been found to be too great.

On horizontal surfaces the gauge is self-centering. On other surfaces it must be firmly held against the measured member and micrometer readings taken by a second party. The pressure applied in so doing should be directly over the contact points. If so applied, there will be no springing of the parts of the gauge. Overhanging handles were not considered in designing the instrument.

In respect to the choice of material, steel was used. Its advantages are a high modulus of elasticity and high elastic limit. Some authorities give high nickel alloys—35 to 38% nickel—an elastic limit above 60 000 lb. per sq. in. The writer's tests on a 30% nickel alloy showed

an elastic limit of less than 20 000 lb. per sq. in., with a modulus of elasticity about 7 000 000 lower than that of steel. Mr. Howard.

The strain gauge is a transfer instrument. Hence, its coefficient of expansion is a negligible matter provided changes in length do not occur with rapidity. No more than 5 sec. need elapse between readings on the measured member and on the standard bar, to which all readings are referred. Within this interval of time the change in length of the strain gauge is not a sensible amount.

A standard bar of annealed steel, with planed surfaces, is used. Forged and rolled shapes commonly retain internal strains of considerable magnitude. They may be nearly coincident in value with the elastic limit of the metal, in which case the shape of the bar is not as stable as desired.

The preparation of the drilled and reamed holes used to define the extremities of the gauged lengths on the measured members is conducted in the manner described in "Notes on the Construction of Ordnance," April, 1887. Strain-gauge measurements made at that early date demonstrated their possibilities as an accurate method of investigation. The results were checked on work, published in June, 1886, having reference to changes in diameters at different depths in the walls of a thick hollow cylinder, in comparison with a mathematical analysis of the expected strains, a rigid check on experimental work which the strain gauge successfully met.

Clyde T. Morris,* M. Am. Soc. C. E. (by letter).—In connection with the work which Mr. Fowler has described in this paper, there are several points which stand out as notable advances in structural engineering: Mr. Morris.

1st.—The determination of the existing dead load stress in a redundant member (the center top chord in this case) of a statically indeterminate structure, thus determining the true dead load stresses in all the members.

2d.—The re-distribution of the dead load stresses in the entire structure to help equalize the unit stresses, by changing the length of the redundant member and thus changing its dead load stress.

3d.—The adjustment, to a uniform initial stress, of the rod bracing by the use of the strain gauge.

The use of the strain gauge in measuring live load stresses in structures is not new, and its adaptation to the revision of this structure has been fully described by Mr. Fowler. There is no doubt that many bridges in use to-day, which are overloaded, could be reinforced so as to give service for many years to come, at a cost of but a fraction of the amount required for a new structure, if proper stress investigations were made and revisions worked out.

* Columbus, Ohio.

Mr. Morris.

A little more detailed description of the application of some of the strain-gauge measurements may not be out of place here.

Sway-Rod Adjustment.—The adjustment of the sway and lateral rods is an operation which is necessary every few years in all structures with adjustable bracing, and the results by the use of the strain gauge are so much more reliable than judgment by sounding with a hammer, that it should always be used in important structures.

In the preparation of the gauge holes for this work the extreme care used in the other parts of the structure was not found necessary, due to the fact that the whole operation for one pair of diagonals would be completed by one observer, with a single instrument, within a period of perhaps an hour. For the same reason no temperature corrections were made.

The gauge holes were made with a center punch ground to a slightly sharper angle than the points of the strain gauge. After a little experience these were found to be quite satisfactory for temporary use.

After the gauge holes were prepared, a set of initial readings would be taken on the two diagonals of a panel and then the turnbuckles slacked off until there was no stress in the rods. This could be determined by observing when the point was reached where further slackening produced no further change in the gauge reading. This reading was taken as a zero reading and then the turnbuckles were tightened until the desired reading on the gauge was obtained. A 20-in. Berry gauge was used. The tension put in the rods was about 7 000 lb. per sq. in. In tightening up the rods care was used to draw up both diagonals evenly so that the symmetry of the panel would be preserved.

The adjustment of the ninety-six sway rods occupied about four days. The writer took two bridge men with him to operate the turnbuckles. No staging was found to be necessary.

Floor-Beam Stresses.—Stress measurements were made with an 8-in. gauge, on both flanges of a floor-beam, at a number of points from the center out, and also on the curved brackets at the ends, and down the posts, to determine the variation in unit stress. Readings were also taken on the post below the highway deck to determine what effect would be carried down from the bending of the railway floor-beams. The results for the post U_2L_2 are given in Table 4, and show that below the highway floor, the unit stresses at Corners 1 and 2 are equal and those at Corners 3 and 4 agree within one division of the gauge. This proves that there was little or no transverse bending carried below the highway floor-beams. It was decided therefore that the reinforcement of the railway floor-beams and vertical posts could be determined by calculating the stresses due to floor-beam bending in the quadrangle frame formed by the two posts, the railway floor-beam, and the highway floor-beam. The variation in the measured unit stresses,

and those calculated by this method agreed very well. The measured unit stresses at the tops of posts U_2L_2 and U_6 L_6 are shown in the diagrams of Fig. 6. Mr. Morris.

The reinforcement of the floor-beams on the 115-ft. approach spans was also figured in this manner, although it is probable that the knee-braces there do not have as much stiffening effect on the floor-beams as the very heavy curved brackets of the main arch span.

Almon H. Fuller,* M. Am. Soc. C. E. (by letter).—The writer has found much of interest in the Niagara Arch revision itself, but that interest has been greater, due to the fact that he went through the fundamental stress work as a thesis at Cornell University in 1898, just after the original structure was completed. He was told by the college authorities that very few structures of that nature had been rigidly analyzed and that the applications of the available theories were not generally understood. R. S. Buck, M. Am. Soc. C. E., makes the statement† that the sections used were means between those given by theory and those required for a three-hinged arch; the statement is repeated in different words by the late L. L. Buck, M. Am. Soc. C. E.‡ The computations in the writer's thesis brought out the fact (now generally recognized) that the successive approximations in stress work and design tend to distribute the material from the upper chords and web members to the lower chords. Mr. Fuller.

The situation in regard to the original design of the Niagara Railway Arch is that an excess of material was put into the upper chords and web members due to too few approximations in the stress work and that the light bottom chords of the original design were detected by the "eternal fitness of things" and increased as stated. The designers "builded better than they knew", as has already been stated by the author, and the structure contained more material than was needed to satisfy the specifications. This is one of the principal reasons why it was possible to increase the present capacity so readily, and, at the same time, it is the reason why such a close scientific study was justified and demanded. These facts were known by the writer and by the author before the revision was made, but it was not known in advance how the excess material was distributed. The determination of this distribution and the re-arrangement of stresses to meet it by means of the toggle adjustment were early recognized by the author and became leading features of scientific interest.

The author's suggestions in regard to wheel loads and column formulas are interesting and valuable. Previous discussion makes it evident that many able supporters will be found for both and just as evident that neither will be adopted without a struggle. Is it not per-

* Ames, Iowa.

† *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 137.

‡ *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 160.

Mr. Fuller.

tinent to suggest, therefore, that the discussion makes it evident that both are live issues and that the interests of the profession will justify the appointment, by this Society, of special committees to consider them? It is gratifying to note that Mr. Schmitt's discussion also suggests committee action on the entire subject of bridge specifications.

In this connection, it is interesting to note Mr. Ammann's reference to secondary stresses. Very few men are as well qualified by ability and accomplishments as Mr. Ammann to discuss this perplexing subject. It seems surprising, therefore, to have him take the position that secondary stresses should be included only in the general factor of safety. Designers in general do not have the insight that Mr. Ammann does into the intensity and distribution of secondary stresses. Even if they did, it would certainly be more scientific and more practical as well, as experience is multiplied, to reduce the element of ignorance in the factor of safety and not only have the assurance that the higher unit stresses urged by the author may safely be used, but the added assurance that the material would be more logically distributed.

Mr. Belzner's inquiry concerning the respective accuracy of the Berry, Howard, and Fuller-West strain gauges is a reminder of Mr. Fowler's request to the writer to present the conclusions from the Niagara work and the subsequent comparison made at Lafayette College. The new instrument which Mr. Fowler has called the Fuller-West gauge, was developed in the shop of the Civil Engineering Department at Lafayette College, as a result of the Niagara work and the discovery of the "Last Word" dial. Such a large proportion of the development, as well as the excellent workmanship, was contributed by Mr. M. L. West, Mechanician for the Department, that the writer has designated it by the simpler and, he thinks, the more appropriate name of the West strain gauge. In speaking of the Howard instrument, Mr. Belzner says:

"In order to secure such results, which are absolutely essential, persistent patience and painstaking thoroughness, combined with good judgment on the part of the observer, are required until he gains perfect control of the instrument, after which he will experience little or no difficulty; and it is upon the observer that the value of such measurements will depend."

This also applies to both of the others, but apparently to a somewhat lesser degree. In other words, the limited experience of the writer in the use of the Howard gauge suggests that greater care in manipulation is required for the same precision than with either of the other instruments and that less speed is probable. He believes, therefore, that most observers would secure results of greater value in a given time with either of the other instruments. The writer prefers the Berry gauge for some work and the West gauge for other. In general, he, and the students he has broken in, will get results of

apparently the same precision in a shorter time with the West gauge; but no doubt those observations have been obscured somewhat by local prejudices. He feels, however, an added security in the work when one is used as a check against the other, and would feel just a little safer with both of them on an important piece of work. Mr. Fuller.

The indication, mentioned on page 1938, that the stress in tension members was greatest toward the center, was also noticed in a thesis investigation made under the direction of the writer by four seniors (Messrs. R. S. Gutelius, F. H. Krick, R. H. Moyer, and J. J. Waters), at Lafayette College in the spring of 1920. The bridge, in this investigation, was a multiple-span, double-track, riveted structure which had just been completed over the Delaware River between Easton, Pa., and Phillipsburg, N. J., by the Central Railway of New Jersey. The data are insufficient, of course, to conclude that these peculiarities in distribution always exist, but they suggest that they may be present and that attention might well be given to such detailing as will insure a more even distribution.

The thesis on the bridge mentioned also brings out the double secondary stresses, as illustrated for the Niagara Arch, on page 1936, but the variation is much smaller; in other words, the distribution is more nearly uniform. Does it not seem probable that the approach to uniformity of distribution in the newer structure is influenced favorably by the exceptionally rigid sway and lateral bracing? The values given on page 1936 for the fiber stresses in each plane at Niagara are evidently the averages of two values. It is apparent, therefore, that greater stresses exist on single corners, and it would be interesting, and, perhaps, valuable, to have them given.

The writer regrets that it has been impracticable for him to make the close study of Mr. Fowler's paper and his own notes which he had intended. As he gets further away from field and office work, the possibilities seem to grow, and he feels that the data gathered at Niagara have values even beyond those which have been brought out, and that they comprise a storehouse which may be drawn on as need develops for that particular kind of information.

Charles Evan Fowler,* M. Am. Soc. C. E. (by letter).—The writer has been much surprised to find how little use, comparatively, has been made of the strain gauge in bridge examination and revision. This is no doubt due to the facts that recent instruments of comparative simplicity have not been extensively tried out in practical work, and that the methods to be followed in order to obtain results are not understood to be only common-sense ones after all. One recent report on the strain-gauge measurements of a structure calls attention to the inaccuracy of readings due to rusting of the gauge Mr. Fowler.

* New York City.

Mr. Fowler.

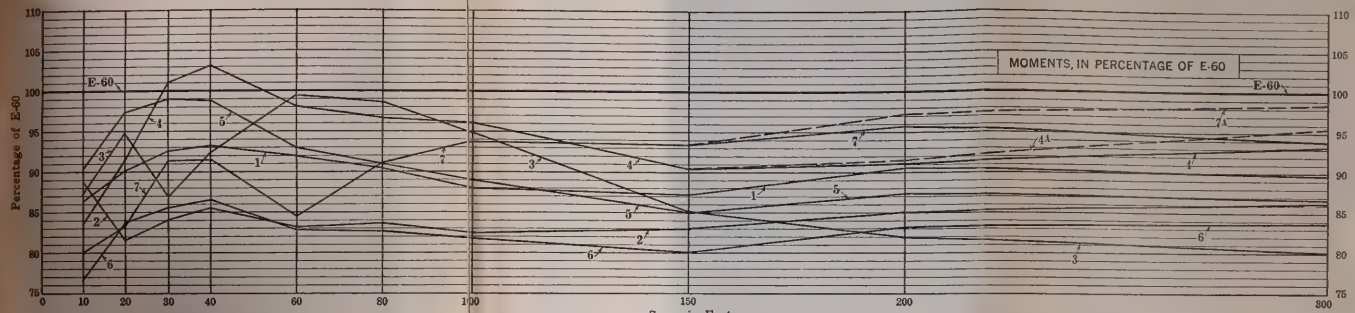
holes, whereas the simple expedient of greasing them would have obviated the trouble.

The use of a mirror instrument is not necessary for the required practical accuracy, provided the observer has the necessary experience, backed up by the proper personality for careful and painstaking work. Although it is much easier for the engineer to tell the owner that his bridge is worn out or has been outgrown by the traffic, often he has no excuse for belonging to the Profession if he allows a good bridge to go into the scrap heap.

The analysis of railway bridge loading is valuable in indicating that it is probably now possible for an agreement to be reached with locomotive builders as to the future limiting weights; having in mind that this may not deter a railway manager from adopting a type of locomotive that is several degrees heavier than the traffic calls for. This is evident from one case where heavy Mallet locomotives were adopted for a low-grade line; when a Mikado with not more than 50 000 lb. on the drivers would have been sufficient.

The scheme of bridge loading proposed by the writer to take the place of the obsolete Cooper system has been somewhat misunderstood, and it was based on the data gathered by Messrs. Welty and Bouton for the American Railway Engineering Association, which were plotted in percentages, as shown in Plates XXXVIII and XXXIX. The calculations were made by including nearly thirty of the very heaviest locomotives of each type in use, and the writer in Table 1 refers to only four of these, or those giving the heaviest moments for each length of span. Thus, no "hand-picking" was done as Mr. Ammann seems to suppose, and it would seem that this scheme would answer the requirements of Dr. Waddell by simply plotting an equivalent load curve based on the same moments. Should any one have the temerity to build a locomotive giving heavier moments, the curve of moments or of equivalent loads could readily be revised from time to time. At any rate, the operating departments of the railways would know at once that no loads now in existence would be too heavy for bridges built for such a loading.

That many of the railway companies are not satisfied to start a design for a bridge with an initial error of from 10 to 15% in the live load stresses, is shown by the fact that Cooper's E-70 loading is being used by many since the adoption of E-60 loading in the new or 1920 American Railway Engineering Association specifications. This is certainly nearer the truth than E-60, being only 5% too light for short spans, but there will probably be considerable metal wasted in spans from 150 to 300 ft. long. Everything developed since 1885 in train loadings has shown the futility of the ultra-refinement of bridge calculation for wheel loads. Moving the locomotives used at



GROUP I

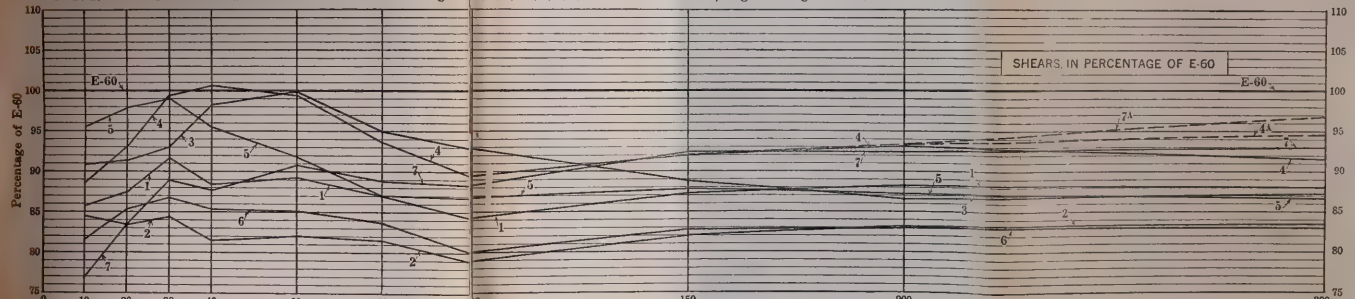
1—U. S. Standard 4-8-2-B and 210 000-lb. cars.
2—U. S. Standard 4-6-2-B and 210 000-lb. cars.
3—U. S. Standard 2-6-6-2-B and 210 000-lb. cars.
4—U. S. Standard 2-10-2-B and 210 000-lb. cars.

4A—U. S. Standard 2-10-2-B and 6 000-lb. per lin. ft. uniform load.
7A—Santa Fe Type 2-10-2 and 6 000-lb. per lin. ft. uniform load.

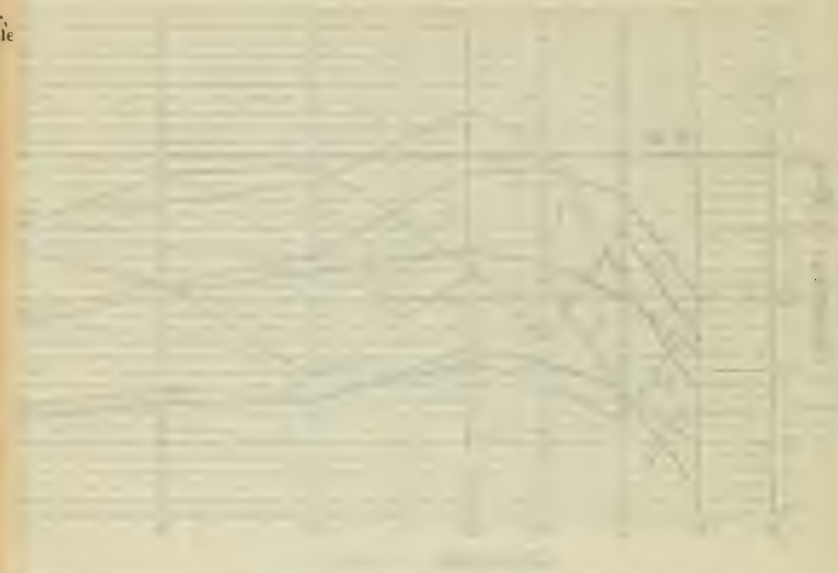
GROUP I (Continued).

5—N. Y. C. R. R.—Mikado—H7A—2-8-2 and 210 000-lb. cars.
6—U. S. Standard 2-8-2 and 210 000-lb. cars.
7—Santa Fe Type 2-10-2 and 210 000-lb. cars.

Engines 1, 2, 4, 4A, 5, 6, 7, and 7A double-headed; Engine 3 single-headed.



Mr.
Fowle



Mr. Fowler.

Niagara from having the first driver over the point, to the second driver, the third driver, or the fourth driver, gave little appreciable difference in truss stresses. Yet, with all said, it is certain that any engineer would soon discontinue the services of an assistant who constantly made direct errors in calculations of from 5 to 15 per cent.

Mr. Ammann's criticism with regard to impact allowance is well taken, from his point of view, but the speed on the Niagara Arch Bridge, had not been gradually decreased as the bridge became more heavily loaded. The speed on the Roebling Suspension Bridge had been limited in the contract with the railway company to 5 miles per hour, and this was still in force on the Arch. However, by sufferance, speeds up to 25 to 30 miles per hour had been allowed. Thus, it was a question as to how much of an increase beyond 5 miles per hour should be allowed in the new contract; and the readings showed that 17 miles was the maximum increase allowable if the bridge was to be brought up fully to E-60 capacity. Due to the condition of the structure and to yard conditions at both ends, it was out of the question to run up to test speeds that would determine the critical speed on the Arch. However, it was certain that at some speed beyond 24 miles, the impact would reach a maximum, then probably drop off, and in the writer's opinion possibly increase at a higher speed up to a second maximum. This would be dependent on many other things than speed, notably the type of bridge and panel length. The data on impact gathered by the American Railway Engineering Association are sufficient, no doubt, to make possible the derivation of a formula for the calculation of the critical speed for ordinary types of bridges, and it is interesting to note that a recent review of these data confirms the findings at the Niagara Arch, except that after reaching a maximum at about 50 miles per hour, the curve drops off and does not again rise for the quite high speeds of the tests. The Profession would be greatly benefited if these were developed fully and published. However, the formula given in the paper may be used with confidence in cases where it is necessary to determine the reduction in speed which is necessary on an old structure.

The writer does not agree with Mr. Ammann as to the omission of secondary stress from the total stresses. The great advantage in calculating the secondary stresses in a new truss design is to determine the worth of the scheme under consideration, and when they are excessive, the design should be modified, if possible. The writer has had numerous verbal discussions which confirm him in the belief that all stresses must be included in the total stresses and that one must gradually approach the elastic limit of the material used for unit stresses, but not go beyond, as some have advised—arguing that the secondary stresses would disappear by adjustment and readjustment.

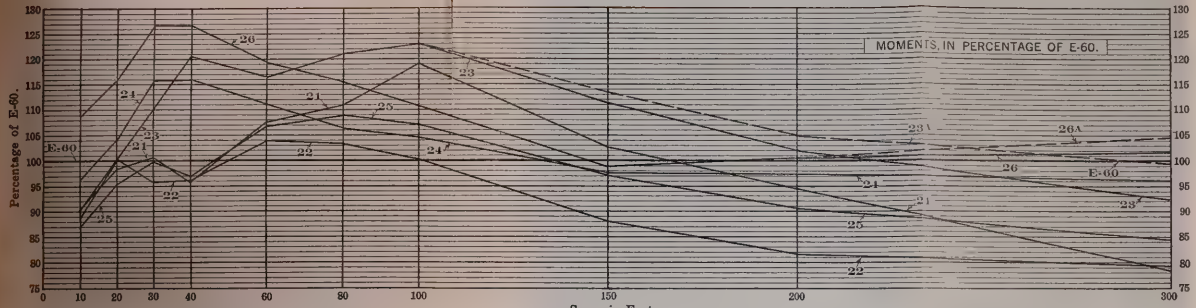
Mr. Fowler.

The column formula used for the Niagara Arch is well on the safe side, but the writer agrees with Dr. Waddell and Mr. Ammann that further study of the subject is advisable; nothing better can be done than has been done until a thorough study and review have been made of all the column tests made during the last thirty years. Indications are that the opinion expressed to the writer recently by George H. Pegram, Past-President, Am. Soc. C. E., is correct, that when this is completed we will return to a formula of the Gordon type as most nearly representing test values for unit stress. This seemingly accords with the findings of O. E. Hovey, M. Am. Soc. C. E., with regard to the report of the Special Committee on Steel Columns and Struts of the Society. Recent specifications do not do justice to the American bridge engineer, who has long prided himself as being thoroughly scientific and not in favor of compromises.

The point made by Mr. Ammann with regard to column design is well taken, and much could be gained by more study in the designing and detailing of columns. The recent column tests by the U. S. Bureau of Standards emphasize the fact that a great difference in column strength is obtained in the arrangement of lacing, as was pointed out long ago by John E. Greiner, M. Am. Soc. C. E., in his study of secondary stresses in columns due to the arrangement of lacing bars. The fact noted by Mr. Schmitt with regard to the Niagara Bridge design being a generation ahead of its time, is especially true of the columns and built tension sections and their detailing, and they are entitled to much study by the present generation of designers.

The comparison of the strain gauges used at the Niagara Bridge was not carried far enough to give conclusive data in answer to Mr. Belzner, but without any reflection on either of the three types that were tried out, it may be said that although the Howard gauge is highly satisfactory for static work as regards final results, it is not at all possible to use it where moving loads are constantly passing over a bridge, nor for impact readings. Neither is it as easy to use as either of the other types. The Berry gauge was highly satisfactory for the dead load readings and for the static live load, but for impact work the vibration of the needle of the Ames dial made the readings too large, which, of course, was on the side of safety. The Fuller-West instrument was, in the writer's opinion, the best for all classes of work, and the only one that was really accurate for impact, as there was no disconcerting vibration of the needle of the "Last Word" dial. For the work done it was more satisfactory than a deflectometer.

The great care necessary in drilling the holes for the gauge points and in holding the strain gauge perpendicular to the member was a vital point, and necessitated a large number of check readings. The new scheme devised by Professor Fuller's mechanician, Mr.



GROUP II.

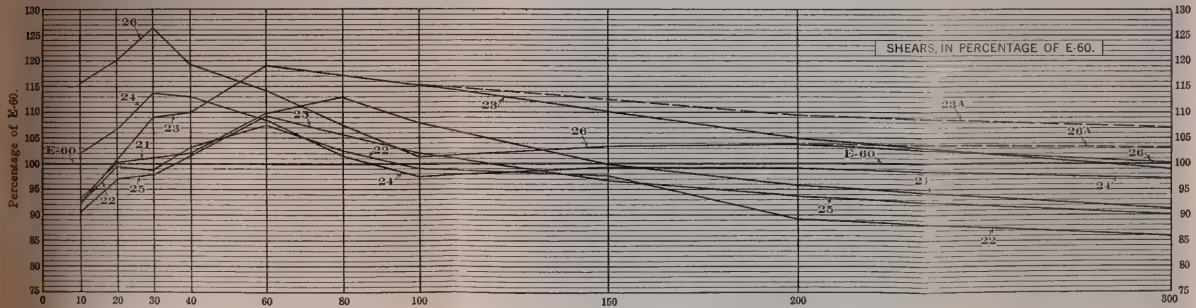
21—The Virginian Ry. 2-8-8-2 and 210 000-lb. cars.
22—The L. S. & M. S. Mallet 0-8-8-0 and 210 000-lb. cars.
23—The Virginian Ry. 2-10-10-2 and 210 000-lb. cars.
24—Erie Engine 2-10-2 and 210 000-lb. cars.

23A—The Virginian Ry. 2-10-10-2 and 6 000-lb. per lin. ft. uniform load.
26A—Pennsylvania Ry. NIS 2-10-2 and 6 000-lb. per lin. ft. uniform load.

GROUP II (Continued).

25—U. S. Standard 2-8-8-2-B and 210 000-lb. cars
26—Pennsylvania Ry. NIS 2-10-2 and 210 000-lb. cars

Engines 21, 22, 23, 23A, and 25, single-headed; Engines 24 and 26, 26A double-headed.



Mr.
Fowler.

West, is one of the greatest improvements yet made for strain gauge work. This consists in drilling the small holes and then setting into them a small ball-bearing ball, as in Fig. 24. The leg of the instrument, instead of being pointed, has a conical recess to set on the projecting half of the ball. The instrument thus gives perfect readings when the legs are considerably out of plumb, and thus a great number of check readings are not required. Mr. Fowler.

Early morning readings before the sun heats up the steel, are not necessary if proper temperature readings are made and the corrections calculated. In fact, it is next to impossible to get test trains except during regular working hours. The advisability of permanent gauge points on important structures mentioned by Mr. Howard should be taken to heart by all builders of big bridges, as the cost would be small, and the gain to the owner beyond question, to say nothing of the scientific data that would eventually be gained. The cost need not exceed $2 per pair of points even if the large holes and the tapping

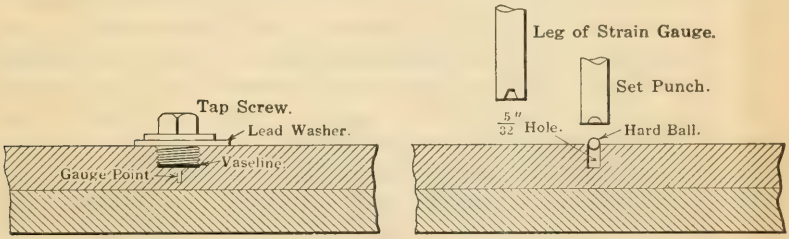


FIG. 24.

are done with a breast hand drill; thus, the points on 150 members of a long span would cost about $300. (Fig. 24.)

The many points raised by Mr. Schmitt are quite vital in the consideration of general and detail design. There can be no question but that the designer in endeavoring to make up his sections for easy future reinforcement will get a much better average make-up of sections; and he should also endeavor to design members that can easily be repainted and re-riveted in the field. One vital point, as brought out by results at the Niagara Bridge, is that all plates and shapes should be thickened about $\frac{1}{32}$ in. for each 20 years of the proposed life of a structure; and where there is apt to be excessive corrosion, then this amount should be doubled. Thus, the exposed top plates or angles of beams and stringers, and the horizontal cover-plates or exposed angles should be increased more than $\frac{1}{8}$ in. for an assumed life of 40 years. The extra good care given the Niagara Arch was very exceptional.

Mr. Fowler.

The methods used in the placing of reinforcing on members subjected to dead load stress were certainly superior to any such method as heating which might injure the steel and could not possibly be uniform throughout the length of a member, nor during the time required for the operation. The erratic distribution of stress over the cross-section of a truss member serves to make uncertain the realization of the computed unit stresses, and points anew to the necessity for a careful consideration of all kinds of secondary stresses. The writer cannot agree that the calculation of the stresses for such a two-hinged arch or other form of indeterminate structure is of necessity obviously correct, or as surely correct as for a simple structure, and it is certainly making assurance doubly sure to measure the dead load stresses as was done initially at the Hell Gate Arch and, latterly, at the Niagara Arch. The general run of the results of the latter for static live load was corroborative of the dead load stresses in other members than the center top chord where the dead load stress was actually measured. This could easily have been proven by using a toggle on some other member, at, however, a much greater cost.

The examination of the copper steel salt-water floor this summer (1920) has happily confirmed the reasons for the use of copper steel and of the enamel coating. The repair of some bad spots in the coating did not comprise more than about 5% of the top area, and was practically the only maintenance cost on the bridge for the year. The copper steel, where exposed, had not rusted to any such degree as is usual with ordinary carbon steel. The recent publication of the good condition of copper steel tie-plates, as compared with ordinary tie-plates, was another pleasing confirmation of the wisdom of the experiment. The cost of renewing some of the troughs, if necessary some years from now, will be only a fraction of the cost of renewing top flanges of beams and stringers at a cost of probably $30 000. The low first cost of such a floor and the saving in dead load on the structure should recommend a similar design for new bridges.

The question as to the rod bracing allowing more vibration in the Arch than would be the case with stiff bracing was carefully studied. It is true that the adjustment of the rods by using the strain gauge did add to the stiffness, as some of them were stressed to the elastic limit and undoubtedly gave more or less under lurching trains, but there was a general swing to the long rods which was accentuated by the inertia of the pin and eight loop eyes at the center crossing. The new stiff bracing, of course, was made heavy enough to allow for the future four-tracking of the bridge, but while the gusset connections may induce some secondary stresses in the truss members, it is also true that in the final analysis they serve to shorten in a

degree the length of the main members and to bind the entire structure more firmly together. Mr. Fowler.

The slowness of the center top chords to take full stress after the change of the shims certainly was due largely to the compressive stress that was taken by the tight stringers, by the 100-lb. rails with tight joints, and, lastly, to a certain readjustment of the center bottom joints, where the gussets had been curved out and some of the rivets cut out, as was shown by the check strain gauge readings. Certain it must be that there were infinitesimal elastic adjustments at other truss joints, and this may still be slowly going on.

The riveting done during 1918 and 1919, referred to by Mr. Bentham, was not because of overload, or train vibration, except as found in some joints in the northwest quarter of the arch, which gets more traffic due to yard switching, but was largely due to poor field riveting when the bridge was built. This was not to be avoided when driving 1-in. rivets by hand, as had to be done in 1896-97. Had they been tested with an 8-lb. sledge, as the writer once knew an inspector to do (!), it could have been found at the time. Redriving them with long gun air riveters, drew the plates more firmly together and loosened adjacent rivets to the extent that in some cases all of the 250 or more rivets in a joint would have to be redriven. The low unit stress first used, 6 500 lb. per sq. in., had prevented the joints from working. Many of the hand-driven rivets had never been upset to fill the holes, and were rusted inside from end to end, although this did not show outside, around the heads, the rivet heads in all cases being tight against the member or plate.

The writer has already replied to Mr. Ammann and others in regard to the query of Mr. Lang, pointing out that the Welty-Bouton data prove more conclusively against the retention of Cooper's loadings, than for their continuance as the standard. The writer has seen too many such questions answered by "time" in the past thirty years, not to await "time's" verdict in this case.

The wasting away of the plates from salt-water corrosion was quite uniform over the entire surfaces, whole layers or laminations being removable; for angles, however, it was in patches and depressions, but also over the entire exposed surfaces.

The careful reader will certainly find the matter of highway loads fully covered under the head of "Highway Floor." The 15-ton truck was of the standard type as to the spacing of axles, wheels, and loads. The writer is not aware of any statement to the effect that the bridge had been revised for E-80 loading, having been made in the paper.

Replying to Mr. Bentham's query about keeping trains going: Only one track was removed at a time; when the first track was complete, traffic was shifted and the other track torn up. The only work extend-

Mr. Fowler.

ing across under the track in use was the replacement of the top floor-beam cover-plates. This reduction in strength of beams was made up by shoring and slow speed of trains.

The writer would feel sure, after the experiences with salt-water corrosion at Niagara, that it would be a reasonably simple matter to correct the effects of salt-water corrosion on the lower parts of the Forth Bridge, and to protect it against further corrosion. Whether or not the rusting has been severe enough seriously to endanger the structure would be interesting to know, but it is to be presumed that the data regarding the original design of the bridge, which have never been made public, would make this a matter to be easily determined.

The whole of the Niagara Bridge investigation and revision, and the discussion of this paper, makes one quite certain that engineers cannot now stop, self satisfied, with the present status of bridge design. There is little incentive, it is true, for anything except machine-made plans; but with the data available from which to formulate a real specification why is it not incumbent upon the Society to fix a standard as high as the materials specifications of the American Society for Testing Materials?

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Paper No. 1461

ARCHED DAMS.

By B. A. Smith, M. Am. Soc. C. E.

With Discussion by Messrs. Chauncy Wernecke, L. R. Jorgensen, William Cain, and B. A. Smith.

Synopsis

A solution of the problem of arch dam analysis is offered which, although not free from objection, avoids the obvious defects of the ordinary cylinder theory. These defects are, that, by the latter theory, the arch thrust is a maximum at the base of the dam where, as a matter of fact, it must vanish and that it vanishes at the crest where, in any ordinary case, the deflection is greatest and, as a consequence, the stress due to the arch thrust must be a maximum. Two cases are dealt with: (1) A complete cylindrical dam or tank of uniform thickness throughout; (2) a complete cylindrical dam or tank of uniformly varying thickness from crest to base. The mathematical investigation of these cases is given in Appendix I. Case 1 is found to be included in Case 2.

It is shown in Appendix II that, provided we neglect the twist in the material arising from the base being fixed while the cylinder is deformed by the water pressure, we can use the solutions obtained for the complete cylinder as an approximation to the solution for the deepest part of a dam built in a valley of any form, provided we replace the ordinary Young's Modulus for the horizontal thrusts by a modulus two-thirds as great. The effect of temperature is con-

sidered, and it is demonstrated by a numerical example in Appendix IV that the stresses due to variations of temperature may be considerably greater than those due to the water pressure.

Tables of Michell's functions are given, which enable the solution of any problem on the design of arched dams of wedge-shaped cross-section, the up-stream surface being a vertical cylinder of constant radius, to be made. The chief labor involved is the solution of a set of four simultaneous equations to determine four arbitrary constants which enter into the expression for the displacement of any point in the vertical generators of the cylinders, but with a 6 or 8-figure calculating machine this can be done in about an hour.

A remarkable feature of Michell's functions is the balance which subsists between them leading to very small resultants when the functions are combined in the course of the work. This renders it necessary to exercise care in using them; in particular it will be found necessary to retain six or seven significant figures in the arbitrary constants, otherwise we shall be led to quite erroneous results. A perusal of the numerical work in Appendix III will show this clearly.

Although at first sight the tables appear to be limited to a height of 30 ft., it will be found that this is not really the case, the numbers 1 to 30 in the tables are merely like the ordinary numbers in a table of logarithms, and the height may be 200 ft., or even more. The limitation of the investigation to the two cases of uniform thickness and a truncated wedge-shaped cross-section will, it is hoped, be viewed with leniency, when account is taken of the somewhat laborious nature of the investigation even in these two comparatively simple types of arched dams.

The Cylinder Theory.

A recent paper by H. Hawgood, M. Am. Soc. C. E., on the Huacal Dam, Sonora, Mexico,* indicates that the subject of the design of arched dams is not yet exhausted. The usual procedure hitherto has been to assume that the arch thrust is given by the expression†:

$$T = p\,R$$

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 564.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), Table 1, p. 567.

in which

p = the pressure, in pounds per square foot, at the section considered;

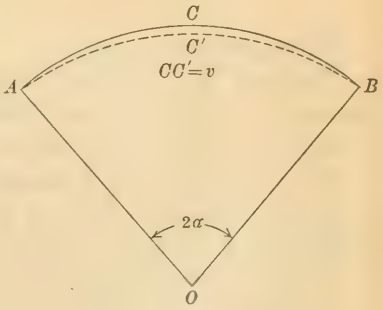
R = the radius of the up-stream face, in feet;

T = the arch thrust, in pounds per foot height of the dam at the given section.

It is, however, generally recognized by engineers that this expression can only give the thrust at points so far above the base that the arch can deflect sufficiently to allow the material to come into compression. In the case of the Huacal Dam, for example, the radial displacement near the base would have to be of the order of $\frac{1}{8}$ in. in order to allow the calculated arch thrust to exist. An elastic movement of the foundation rock of this magnitude is hardly conceivable. The assumption that the base is rigidly fixed is probably a much nearer approximation to the actual conditions.

Proposed Modification of the Cylinder Theory.

This paper is an attempt to carry the approximation of the cylinder theory a step further by taking account of the shear across a horizontal plane and the corresponding bending moment about a horizontal tangent to the cylinder.



PLAN OF SEGMENTAL DAM
Fig. 1.

First, consider a complete cylinder symmetrically deformed so that the displacement of every point is radial. It is clear that, in this case, there will be no shear across vertical axial planes, and as a consequence there will be no bending moment about the vertical generators; that is, in the case of the complete cylinder, bending as a horizontal arch does not occur.

Segmental Dams.

Next, consider a segment of the cylinder $A\ C\ B$, Fig. 1, between two vertical generators at A and B, respectively, subtending an angle $A\ O\ B = 2\ \alpha$ at the axis O, and suppose this segment is free to slide on the base and is merely supported at the ends A and B so that the bending moment vanishes at each of these ends. Then,

assuming that the bending moment vanishes throughout and that the segment is subjected to a uniform compressive stress (arch thrust) throughout its length so that the elastic compression, f, is constant throughout, the segment deforms to the shape $A\ C'\ B$ and, as is shown in Appendix II, the displacement $C\ C'$ $(= U)$ of the center of the segment is given by

$$U = \frac{f\ R\ \alpha\ (1 - \cos.\ \alpha)}{\sin.\ \alpha - \alpha \cos.\ \alpha}$$

where

$f =$ the elastic compression of the material; that is, the ratio of the contraction of any given length under compression to the original length of that portion;

$R =$ the radius of the cylinder;

$2\alpha =$ the angle $A\ O\ B$.

If $2\ \alpha$ is less than $\frac{2\ \pi}{3}$, or 120°, there can be written, approximately,

$$U = \frac{3}{2}\ f\ R$$

which is independent of α.

This means that the radial displacement U at C, the center of the segment, $A\ C\ B$, supposed to be fixed in position at the abutments A and B, though free to turn about these points, is the same as it would have been if $A\ C\ B$ formed part of a complete cylinder, subject to the same stress, the value of Young's Modulus E for which is two-thirds of that of the actual material.

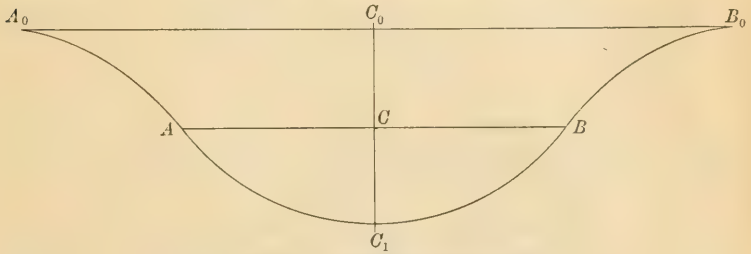
It may be mentioned, to anticipate the objection being taken, that unless extraordinary precautions are taken to fix in direction the ends of the arch, A and B, there will be deflections at these points which will correspond very nearly to the deflections due to no bending moment at A and B.

Dams Constructed in Valleys of Any Profile.

In order to pass from the ideal case to the actual one, namely, a dam built in a valley of any profile, the twist in the surface which occurs when the base, instead of being free to slide, is fixed throughout, is neglected. If this is done, $A\ C\ B$, Fig. 2, may be regarded as any horizontal section of a dam of which $A_0\ C_0\ B_0$ is the crest and $C_0\ C\ C_1$

the central vertical generator so that $A_0\ A\ C_1\ B\ B_0$ is the profile of the valley which, under the conditions premised, may be of any shape whatever, as long as it is approximately symmetrical. As shown in Appendix II, and subject to the limitations there given, the radial displacement of any point C of the central vertical generator $C_0\ C\ C_1$ will then be the same as if that generator formed part of a complete cylinder the value of Young's Modulus for which is two-thirds of that of the actual material of the dam.

The solution obtained for the complete cylinder is thus applicable (subject to the reservations mentioned above) to the case of any ordinary arched dam, the value of Young's Modulus being taken as two-thirds of its value for the material of which the dam is built.



ELEVATION OF SEGMENTAL DAM IN VALLEY

FIG. 2.

LIMITATION OF SOLUTIONS TO TWO TYPES OF DAM.

Two cases are here considered: (1), a cylindrical dam or tank of uniform thickness throughout; (2), a cylindrical dam of finite crest width and of uniformly varying thickness from crest to base.

The difficulty of obtaining a solution of the equation of equilibrium of the material in other cases must be pleaded in excuse for no attempt being made at a general solution applicable to dams of any cross-section; the simple cases considered are troublesome enough. The solution of the first case, the cylindrical dam of uniform thickness, results quite simply in terms of known functions, circular and hyperbolic. The solution of the second case is obtained in series. Four different series each of which satisfies the equation of equilibrium are required; these are given in algebraic form, together with tables for the values of each of the functions and their first three derivatives, which are all required to satisfy the boundary conditions at the

crest and at the base of the dam. These functions will be referred to as Michell's* functions.

It is shown that the solution of the fundamental equation can be formally expressed in terms of Bessel's functions, but as the functions involved have complex arguments it is considered preferable to leave the solution in terms of Michell's functions, which involve only a real argument. The solution in terms of Bessel's functions, however, is useful as it permits the extension of the tables by using the "semi-convergent" expressions, and this makes it possible to deduce the solution for the uniform dam from the case where the thickness is variable.

An equation corresponding to the fundamental equation of Case (2) for a particular case, namely, when the section is a triangle with its crest at the water level, is given by H. Lorenz†, but as this solution is of very limited application and as only two solutions of the four series of the fundamental equation are given, and then only in series form, it cannot be regarded as a complete solution of the problem.

The solution obtained for Case (1) shows that the bending moment rapidly dies out as we leave the base of a dam or tank of uniform thickness, whether the base is fixed in direction or merely simply supported, and that at a height above the base equal to three or four times the thickness of the dam the ordinary cylinder formula, $T = p\,R$, applies quite nearly enough. This statement must not be taken to indicate that the bending moment near the base can be disregarded, for this cannot be done altogether, but it shows the great assistance given by the arch in supporting the load on the dam.

Temperature Effect.

The effect of temperature changes on the stresses is considered, and it is shown that a very moderate range of temperature will give rise to stresses considerably greater than those due to the water pressure. As a consequence, it appears that, if practicable, a concrete dam should be built at a time when the temperature is well below the mean temperature of the locality, for if this is done the effect of a rise of temperature is to enhance the arch effect, and it may even cause the dam to deflect up stream against the water pressure. This

* These functions are here published for the first time.

† "Lehrbuch der Technischen Physik", Vol. IV, p. 618.

was found to happen in the case of the Wooling Dam, to which these calculations were first applied.

A similar phenomenon is suggested by the observations of the deflections of the Barren Jack Arch Dam shown in Fig. 6 of the paper* by Mr. Hawgood.

The Arguments of Michell's Functions.

Attention should be called to the fact that the arguments x in the tables of Michell's functions are not actual heights but are mere numbers like the arguments in a table of logarithms. The relation between x and the actual depth z below the origin, which is taken to be at the intersection of the up-stream and down-stream faces of the dam, is given by

$$z = a\,x$$

where

$$a^2 = \frac{1}{12}\,c^2\,R^2\,\frac{E_1}{E_0}$$

in which

c = the tangent of the angle between the up-stream and down-stream faces of the dam, in a vertical axial section;

R = the radius of the cylindrical up-stream face;

E_1 = Young's Modulus for the vertical forces;

E_0 = Young's Modulus for the arch thrust.

In the case of the complete cylinder, E is the ordinary value of Young's Modulus. In the case of the segmental dam, E_0 becomes $\frac{2}{3}E_0$. The value of E_1 depends on the vertical reinforcement. If p is the ratio of area of steel reinforcement to concrete, then

$$E_1 = E\,(1 + 15\,p)$$

where E is the ordinary Young's Modulus for concrete for the vertical reinforcement. For steel, $E_1 = 30 \times 10^6$ lb. per sq. in. $= 15 \times E$.

For example, if $p_1 = 2\%$ and $E = 2 \times 10^6$ lb. per sq. in. and $p_0 = 1\%$ for the horizontal reinforcement,

$$E_1 = 2 \times 10^6\ (1 + 15 \times 0.02) = 2.6 \times 10^6$$
$$E_0 = 2 \times 10^6\ (1 + 15 \times 0.01) = 2.3 \times 10^6$$

The fact that the dam is segmental does not affect E_1, in calculating which the ordinary Young's Modulus is used. Suppose, for

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 578.

example, a segmental dam of 200 ft. radius with the up-stream face vertical and the batter of the down-stream face 1 in 6, so that $c = \frac{1}{6}$, and with 2% vertical reinforcement and no horizontal reinforcement. There is obtained

$$E_1 = 2.6 \times 10^6 \text{ lb. per sq. in.}$$
$$E_0 = 2.0 \times 10^6 \text{ lb. per sq. in.}$$

and since the dam is segmental, replace E_0 by $\frac{2}{3}\ E_0$, that is, by $\frac{2}{3} \times 2.0 \times 10^6$, and

$$a = \frac{1}{6} \times 200 \times \sqrt{\frac{1}{12} \times \frac{2.6 \times 10^6}{\frac{2}{3} \times 2.0 \times 10^6}}$$
$$= \frac{200}{6} \times \sqrt{\frac{1}{12} \times \frac{3}{2} \times 1.3}$$
$$= 13.5, \text{ say.}$$

Now, suppose that the crest width of the dam is 4 ft. 6 in. and its depth 202 ft. 6 in. (corresponding to a dam about 200 ft. deep). Since the batter is 1 in 6, the origin is at a height $6 \times 4.5 = 27$ ft. above the crest. At crest level $z = z_0 = 27$ ft. and

$$z = a\,x = 13.5\,x$$

hence

$$x_0 = \frac{27}{13.5} = 2.0$$

At the base, $z = z_1 = 27 + 202.5 = 229.5$ ft., and therefore

$$x_1 = \frac{229.5}{13.5} = 17.0$$

and the tabular values to be used are those corresponding to $x = 2$ at the crest, and to $x = 17$ at the base.

The range covered by the tables will be found to be sufficient for most practical purposes. If it is desired to go beyond the tables, it is suggested that the semi-convergent expressions be used in preference to the series, which become unmanageable at about $x = 30$.

Proposed Method of Design, Using Michell's Functions.

It is suggested that as a convenient method of design the following procedure be adopted:

1.—Determine the maximum flood height above the crest of the dam $= h$, say.

2.—Assume that the pressure at any depth d below the crest is

$$p = w\,(d + h)$$

where $w = 62.5$ lb. per sq. ft. per foot depth (which is of course the same as the weight of 1 cu. ft. of water).

h = height of flood level above crest, in feet.

d = depth of the given point below the crest, in feet.

This will give pressures near the crest slightly in excess of the actual pressures, but this error may be disregarded.

3.—Fix the maximum compressive stress (arch thrust) allowable in the material = K lb. per sq. ft., say, and determine the corresponding thickness of the dam by the ordinary cylinder formula

$$T = p\,R$$

where T = the resultant ring thrust per foot height.

p = the pressure, in pounds per square foot, as given above.

R = the radius of the arch.

The smaller R is, of course, the smaller T will be and the more effective the arch action. If t is the thickness of the arch, in feet, then

$$T = K\ t = p\ R$$

4.—Fix the minimum safe thickness at the crest of the dam in view of possible injury by floating timber or other bodies, say, t_0 ft. Then, if the thickness is to vary uniformly from t_0 at the crest to t_1 ft. at the base in the depth, D,

$$c = \frac{t_1 - t_0}{D}$$

5.—Decide whether the base of the dam is to be rigidly built into the foundation rock by means of steel reinforcing bars sunk well into the rock below, that is, encastrée, or whether the base is to be simply supported by the pressure of the rock down stream against the down-stream face of the dam. The former method will ensure a water-tight base, but can only be adopted when the rock is quite solid. If the latter method is adopted, observe that the vertical generators of the cylinder will deflect down stream under the water pressure, and cause the base of the dam to lift off its seat on the up-stream side causing liability to leakage under the seat which must be provided against by layers of clay and sand or otherwise according to judgment. If it is decided

that the base shall be encastrée, the conditions to be satisfied at the base are:

(1), the radial displacement must be zero, that is, $u = 0$;

(2), the generating lines of the cylinder must remain fixed in direction, that is, $\frac{d\,u}{d\,x} = 0$.

If the base is simply supported, the bending moment at the base vanishes and, as before, the radial displacement must vanish, that is, (1) $u = 0$; and (2) $\frac{d^2\,u}{d\,x^2} = 0$. At the crest the boundary conditions are the same in either case, namely, the bending moment and the shear, both of which are due to the action of the material above the section or that below it, must vanish, since there is no material above the crest, that is, at the crest

$$\frac{d^2\,u}{d\,x^2} = \frac{d^3\,u}{d\,x^3} = 0$$

These four conditions give rise to four equations among the four arbitrary constants which enter into the solution. When these four constants are determined it is possible, by means of the tables, to find the displacement and consequently the arch thrust at a series of points, also the bending moment, the shear and, if necessary, the deflection, although this is of no interest to us except at the base of the dam. When the bending moments have been determined at the points corresponding to the integral values of x given in the tables, a smooth curve can be drawn through these points to give nearly enough the bending moment at any point of the dam. The necessary reinforcement can then be found by the usual rules.

A remarkable feature of the solution is the smallness of the resulting bending moment compared with the magnitude of the four component terms which contribute to it. An example of the numerical calculations is given in Appendix III.

6.—Determine the temperature range to which the dam will be subject and estimate as nearly as practicable the amounts by which the maximum and minimum temperatures vary from the mean temperature during construction. Let e = the coefficient of expansion of the concrete, or 0.0000055 per degree Fahrenheit, say (about $\frac{3}{8}$ in. per chain for 100° Fahr., as against about $\frac{1}{2}$ in. per chain for 100° Fahr. for steel). For satisfactory construction a nickel-iron alloy having

the same coefficient of expansion as concrete, is a desideratum, as this would minimize the internal stresses due to unequal expansion and contraction of the iron and the concrete. Suppose θ is the maximum difference of temperature in degrees Fahrenheit to be expected from the mean temperature observed during construction, regarded as plus if above this mean and minus if below it. As before, let R be the radius of the dam. If now the dam, supposed to be empty, is free to expand throughout, the displacement at any point will be, in feet,

$$v = - e \theta R$$

the minus sign being used to indicate that the points considered move away from the axis. If, after this expansion, the base is now forcibly displaced toward the axis through the distance, v, the generators being held fixed in direction $\left(\frac{d u}{d x} = 0\right)$, if the base is encastrée, or allowed to deflect so that the bending moment vanishes $\left(\frac{d^2 u}{d x^2} = 0\right)$, if the base is simply supported. In either case, the conditions at the crest are that the bending moment and the shear both vanish; that is,

$$\frac{d^2 u}{d x^2} = \frac{d^3 u}{d x^3} = 0$$

If these four conditions are satisfied the dam is clearly in the same condition as if the base had been fixed while the temperature rose by the amount θ.

This again gives rise to four simultaneous equations for four new arbitrary constants. The coefficients on the left-hand side of the equations are the same as those which were used in determining the constants for the water-pressure equations, so that the labor of the solutions is lightened in this case. The stresses due to the temperature are of course to be superposed on those due to the water pressure. An example of the numerical work is given in Appendix IV.

APPENDIX I.

INVESTIGATION OF STRESSES IN CYLINDERS

Explanation of Symbols.

Consider the origin O to be at the top-water level (flood level) of the dam in cases where the thickness is uniform and at the intersection of the up-stream and down-stream faces when the dam is of triangular section, and consider the axis of Z vertical downward. Let

T = ring thrust per unit depth at any point P at depth z below the origin, reckoned positive when compression;

F = the resultant traction or shearing force per unit length along the dam at P exerted by the material above P on that below P, reckoned positive when this traction is exerted toward the axis of the cylinder;

M = the couple about a horizontal tangent at P per unit length of the dam, reckoned positive when the bending tends to make the generators of the cylinder concave toward its axis.

u = the displacement, along the radius, of a point P on the cylindrical surface, reckoned positive when toward the axis of the cylinder. (Strictly speaking, these should refer throughout to the neutral surface, but it is simpler to deal with the up-stream cylindrical surface. So long as the dam is thin the error is not worth considering);

$\frac{d\,u}{d\,z}$ = the inclination of the generator to the vertical, reckoned positive when the lower part is inclined toward the axis;

$\frac{d^2\,u}{d\,z^2}$ = the curvature of the generator at P after strain, reckoned positive when the concavity is toward the axis of the cylinder;

t = the thickness of the dam at P;

I = the moment of inertia of a horizontal section of the dam at P, of unit length along the cylinder, or $I = \frac{1}{12}\,t^3$;

E_0 = Young's Modulus for the horizontal direction. If p_0 is the ratio of the area of horizontal reinforcement to the area of the concrete in a vertical section, assuming that Young's Modulus for steel is 15 times as great as for

ordinary concrete for which it has the value E, then $E_0 = E\ (1 + 15p_0)$;

E_1 = Young's Modulus for the vertical direction. If p_1 is the ratio of the area of vertical reinforcement to the area of concrete in a horizontal section, then, as above, $E_1 = E\ (1 + 15p_1)$;

C_1 = the rigidity of a vertical slab of unit length along the dam; neglecting the change of curvature in the horizontal plane, $C_1 = E_1 I = \frac{1}{12}\ E_1\ t^3$;

p = the water pressure, in pounds per square foot, at $P = w\ (d + h)$, where w is the weight of 1 cu. ft. of water = 62.5 lb., d the depth of the point P below the crest of the dam and h the flood height;

R = the radius of the cylindrical up-stream surface of the dam;

c = the tangent of the angle between the up-stream and down-stream faces.

a = a factor connecting vertical distances with the arguments of Michell's functions defined by Equations (31) and (33) following:

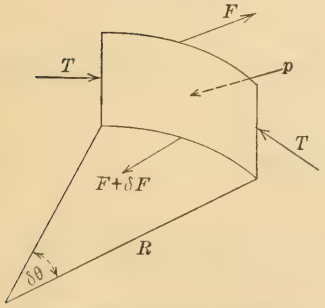
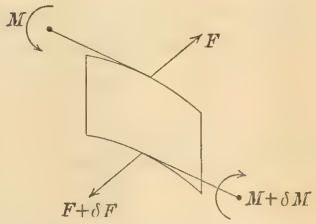


DIAGRAM OF FORCES ON AN ELEMENT

FIG. 3.



SHEARING FORCES AND COUPLES ON AN ELEMENT

FIG. 4.

EQUATIONS OF EQUILIBRIUM

Considering the equilibrium of a small curved rectangular element of height δz and width $R\,\delta\,\theta$, there is obtained (see Figs. 3 and 4), for the forces

$$(F + \delta F)\ R\ \delta\ \theta - F\ R\ \delta\ \theta - T\ \delta\ z\ \delta\ \theta + p\ R\ \delta\ \theta\ \delta\ z = 0$$

and for the couples

$$(M + \delta\ M)\ R\ \delta\ \theta - M\ R\ \delta\ \theta + F\ R\ \delta\ \theta\ \delta\ z = 0$$

Hence, dividing out by $R\,\delta\,\theta\,\delta\,z$

$$\frac{d\,F}{d\,z} - \frac{T}{R} + p = 0 \quad \ldots\ldots(1)$$

$$\frac{d\,M}{d\,z} + F = 0 \quad \ldots\ldots(2)$$

and for the equation connecting the bending moment with the change of curvature, as usual

$$M = C_1 \frac{d^2\,u}{d\,z^2} \quad \ldots\ldots(3)$$

neglecting the bending about the vertical generators, which is very small.

Again, the equation connecting the displacement with the ring thrust in the complete cylinder is

$$T = \frac{E_0\,t}{R} \times u \quad \ldots\ldots(4)$$

and in the case of a segmental dam (see Appendix II),

$$T = \frac{2}{3}\,\frac{E_0\,t}{R} \times u \quad \ldots\ldots(5)$$

Eliminating F between Equations (1) and (2)

$$\frac{d^2\,M}{d\,z^2} + \frac{T}{R} = p \quad \ldots\ldots(6)$$

Substituting for M from Equation (3) and for T from Equation (4), for the case of the complete cylinder

$$\frac{d^2}{d\,z^2}\left(C_1 \frac{d^2\,u}{d\,z^2}\right) + \frac{E_0}{R^2} \cdot t\,u = p \quad \ldots\ldots(7)$$

and in the case of the segmental dam

$$\frac{d^2}{d\,z^2}\left(C_1 \frac{d^2\,u}{d\,z^2}\right) + \frac{2}{3} \cdot \frac{E_0}{R^2}\,t\,u = p \quad \ldots\ldots(8)$$

Equations (7) and (8) are identical except that in Equation (8) the value $\frac{2}{3}\,E_0$ is used instead of E_0; that is, for the horizontal stresses a modulus equal to two-thirds of the ordinary Young's Modulus is applied.

CASE 1.—Dam of Uniform Thickness.

For the case of a dam of uniform thickness, t is constant, so that Equation (7) becomes

$$\frac{d^4\,u}{d\,z^4} + \frac{E_0\,t}{C_1\,R^2}\,u = \frac{p}{C_1} \quad \ldots\ldots(9)$$

or calling

$$\mu^4 = \frac{E_0\, t}{C_1\, R^2} = 12\, \frac{E_0}{E_1}\, \frac{1}{t^2\, R^2}$$

Equation (9) may be written in the form

$$\frac{d^4\, u}{d\, z^4} + \mu^4\, u = \frac{p}{C_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Now, if p involves no power of z above cubes (in this case $p = w\, z$, so that it only involves the first power of z), the particular integral of Equation (10) is

$$u = \frac{p}{\mu^4\, C_1} = \frac{p\, R^2}{E_0\, t} = \frac{w\, R^2}{E_0\, t}\, z \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

for this makes $\frac{d^4\, u}{d\, z^4} = 0$ and, therefore, satisfies Equation (10).

There must now be found a solution of

$$\frac{d^4\, u}{d\, z^4} + \mu^4\, u = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

involving four arbitrary constants (the complementary function).

The auxiliary equation is

$$m^4 + \mu^4 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

from which

$$m^2 = \pm\, \mu^2\, \sqrt{-1} = \pm\, i\, \mu^2$$

hence

$$m = \pm\, \mu\, \sqrt{\pm\, i} = \pm\, \frac{\mu}{\sqrt{2}}\, (1 \pm i)$$

the four roots of Equation (13) are therefore

$$m_1 = +\, \frac{\mu}{\sqrt{2}}\, (1 + i)$$

$$m_2 = -\, \frac{\mu}{\sqrt{2}}\, (1 + i)$$

$$m_3 = +\, \frac{\mu}{\sqrt{2}}\, (1 - i)$$

$$m_4 = -\, \frac{\mu}{\sqrt{2}}\, (1 - i)$$

so that

$$m_1^4 = m_2^4 = m_3^4 = m_4^4 = -\, \mu^4 \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The complementary function is therefore

$$u = A^1 e^{m_1 z} + B^1 e^{m_2 z} + C^1 e^{m_3 z} + D^1 e^{m_4 z} \quad \ldots\ldots (15)$$

This is easily verified by differentiating Equation (15) four times

$$\begin{aligned}\frac{d^4 u}{d z^4} &= m_1^4 A^1 e^{m_1 z} + m_2^4 B^1 e^{m_2 z} + m_3^4 C^1 e^{m_3 z} + m_4^4 D^1 e^{m_4 z} \\ &= -\mu^4 [A^1 e^{m_1 z} + B^1 e^{m_2 z} + C^1 e^{m_3 z} + D^1 e^{m_4 z}] \\ &= -\mu^4 \times u\end{aligned}$$

So that

$$\frac{d^4 u}{d z^4} + \mu^4 u = 0$$

Since

$$e^{i \frac{\mu z}{\sqrt{2}}} = \cos. \frac{\mu z}{\sqrt{2}} + i \sin. \frac{\mu z}{\sqrt{2}}$$

and

$$e^{-i \frac{\mu z}{\sqrt{2}}} = \cos. \frac{\mu z}{\sqrt{2}} - i \sin. \frac{\mu z}{\sqrt{2}}$$

Now, Equation (15) may be written

$$\begin{aligned}u = e^{\frac{\mu z}{\sqrt{2}}} &\left[A^1 \left(\cos. \frac{\mu z}{\sqrt{2}} + i \sin. \frac{\mu z}{\sqrt{2}} \right) + C^1 \left(\cos. \frac{\mu z}{\sqrt{2}} - i \sin. \frac{\mu z}{\sqrt{2}} \right) \right] \\ + e^{-\frac{\mu z}{\sqrt{2}}} &\left[B^1 \left(\cos. \frac{\mu z}{\sqrt{2}} - i \sin. \frac{\mu z}{\sqrt{2}} \right) + D^1 \left(\cos. \frac{\mu z}{\sqrt{2}} + i \sin. \frac{\mu z}{\sqrt{2}} \right) \right] \\ = e^{\frac{\mu z}{\sqrt{2}}} &\left[(A^1 + C^1) \cos. \frac{\mu z}{\sqrt{2}} + i (A^1 - C^1) \sin. \frac{\mu z}{\sqrt{2}} \right] \\ + e^{-\frac{\mu z}{\sqrt{2}}} &\left[(B^1 + D^1) \cos. \frac{\mu z}{\sqrt{2}} - i (B^1 - D^1) \sin. \frac{\mu z}{\sqrt{2}} \right]\end{aligned}$$

or, using new arbitrary constants

$$u = e^{\frac{\mu z}{\sqrt{2}}} \left[A \cos. \frac{\mu z}{\sqrt{2}} + B \sin. \frac{\mu z}{\sqrt{2}} \right] + e^{-\frac{\mu z}{\sqrt{2}}} \left[C \cos. \frac{\mu z}{\sqrt{2}} + D \sin. \frac{\mu z}{\sqrt{2}} \right]$$

The complete solution of Equation (10) is therefore

$$\left.\begin{aligned}u = \frac{p}{E_0 t} R^2 &+ e^{\frac{\mu z}{\sqrt{2}}} \left[A \cos. \frac{\mu z}{\sqrt{2}} + B \sin. \frac{\mu z}{\sqrt{2}} \right] \\ &+ e^{-\frac{\mu z}{\sqrt{2}}} \left[C \cos. \frac{\mu z}{\sqrt{2}} + D \sin. \frac{\mu z}{\sqrt{2}} \right]\end{aligned}\right\} \ldots\ldots (16)$$

From this Equation (16), if $p = w\ (z + h)$, there is obtained

$$\left.\begin{aligned}
\frac{d\,u}{d\,z} &= w\,\frac{R^2}{E_0\,t} \\
&+ \frac{\mu}{\sqrt{2}}\,e^{\frac{\mu z}{\sqrt{2}}}\left[(A+B)\cos.\frac{\mu z}{\sqrt{2}} - (A-B)\sin.\frac{\mu z}{\sqrt{2}}\right] \\
&- \frac{\mu}{\sqrt{2}}\,e^{-\frac{\mu z}{\sqrt{2}}}\left[(C-D)\cos.\frac{\mu z}{\sqrt{2}} + (C+D)\sin.\frac{\mu z}{\sqrt{2}}\right] \\
\frac{d^2 u}{d\,z^2} &= \mu^2\,e^{\frac{\mu z}{\sqrt{2}}}\left[B\cos.\frac{\mu z}{\sqrt{2}} - A\sin.\frac{\mu z}{\sqrt{2}}\right] \\
&- \mu^2\,e^{-\frac{\mu z}{\sqrt{2}}}\left[D\cos.\frac{\mu z}{\sqrt{2}} - C\sin.\frac{\mu z}{\sqrt{2}}\right] \\
\frac{d^3 u}{d\,z^3} &= -\frac{\mu^3}{\sqrt{2}}\,e^{\frac{\mu z}{\sqrt{2}}}\left[(A-B)\cos.\frac{\mu z}{\sqrt{2}} + (A+B)\sin.\frac{\mu z}{\sqrt{2}}\right] \\
&+ \frac{\mu^3}{\sqrt{2}}\,e^{-\frac{\mu z}{\sqrt{2}}}\left[(C+D)\cos.\frac{\mu z}{\sqrt{2}} - (C-D)\sin.\frac{\mu z}{\sqrt{2}}\right] \\
\frac{d^4 u}{d\,z^4} &= -\mu^4\,e^{\frac{\mu z}{\sqrt{2}}}\left[A\cos.\frac{\mu z}{\sqrt{2}} + B\sin.\frac{\mu z}{\sqrt{2}}\right] \\
&- \mu^4\,e^{-\frac{\mu z}{\sqrt{2}}}\left[C\cos.\frac{\mu z}{\sqrt{2}} + D\sin.\frac{\mu z}{\sqrt{2}}\right]
\end{aligned}\right\} \quad (17)$$

Multiplying both sides of Equation (16) by μ^4 and adding to Equation (17)

$$\frac{d^4 u}{d\,z^4} + \mu^4\,u = \mu^4\,\frac{p}{E_0}\,\frac{R^2}{t} = \frac{p}{C_1} \quad \ldots\ldots\ldots\ldots\ldots (18)$$

which is Equation (10), thus verifying Equations (16) and (17).

The arbitrary constants A, B, C, and D are determined by the end conditions at the crest and at the base of the dam.

Dam Free to Slide on Base.

As an example, suppose that the base is free to slide on the foundations. Then at the crest $M = F = 0$; that is, by Equations (3) and (4)

$$\frac{d^2 u}{d\,z^2} = \frac{d^3 u}{d\,z^3} = 0$$

and also at the base $M = F = 0$.

From Equations (2), (3), and (17), using the suffix 0 for the crest and suffix 1 for the base, there is found

$$\left(\frac{d^2 u}{d z^2}\right)_0 = \left(\frac{d^3 u}{d z^3}\right)_0 = \left(\frac{d^2 u}{d z^2}\right)_1 = \left(\frac{d^3 u}{d z^3}\right)_1 = 0$$

Now $z = 0$ at the crest and $z = d$ at the base. Hence, putting $z = 0$ in the second and third of Equations (17) and equating to zero,

$$B - D = 0$$
$$-(A - B) + (C + D) = 0$$

and, using the conditions at the base, putting $z = d$ in the same pair of Equations (17)

$$B e^{\frac{\mu d}{\sqrt{2}}} \cos. \frac{\mu d}{\sqrt{2}} - A e^{\frac{\mu d}{\sqrt{2}}} \sin. \frac{\mu d}{\sqrt{2}} - D e^{-\frac{\mu d}{\sqrt{2}}} \cos. \frac{\mu d}{\sqrt{2}} + C e^{-\frac{\mu d}{\sqrt{2}}} \sin. \frac{\mu d}{\sqrt{2}} = 0$$

$$-A e^{\frac{\mu d}{\sqrt{2}}} \left(\cos. \frac{\mu d}{\sqrt{2}} + \sin. \frac{\mu d}{\sqrt{2}}\right) + B e^{\frac{\mu d}{\sqrt{2}}} \left(\cos. \frac{\mu d}{\sqrt{2}} - \sin. \frac{\mu d}{\sqrt{2}}\right)$$
$$+ C e^{-\frac{\mu d}{\sqrt{2}}} \left(\cos. \frac{\mu d}{\sqrt{2}} - \sin. \frac{\mu d}{\sqrt{2}}\right) + D e^{-\frac{\mu d}{\sqrt{2}}} \left(\cos. \frac{\mu d}{\sqrt{2}} + \sin. \frac{\mu d}{\sqrt{2}}\right) = 0$$

Hence

$$A = B = C = D = 0$$

and the solution becomes simply

$$u = \frac{w R^2}{E_0 t} \times z \qquad (19)$$

hence, by means of Equation (4),

$$T = \frac{E_0 t}{R} \times u = w R z = p R \qquad (20)$$

which is the ordinary cylinder formula.

The condition which must be satisfied for this formula to be applicable is, therefore, that the dam is free to move, or slide, on its foundations.

Dam with Base Encastrée.

As another example, suppose that the base is fixed and encastrée, then at the base the conditions to be satisfied are

$$u = \frac{d u}{d z} = 0$$

In this case it will simplify the work to take the origin at the base and the axis of Z vertical upward.

The fundamental Equation (10) is then unaltered in form, but the pressure is given by

$$p = w\,(d - z) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The solution, Equation (16), remains the same in form but with p as given in Equation (21) and hence $\frac{d\,p}{d\,z} = -\,w$, so that in the first of Equations (17) the sign of w must be altered from + to —. In order to determine the constants, apply the conditions that

when $z = 0$, $\qquad u = \frac{d\,u}{d\,z} = 0$

when $z = d$, $\qquad M = F = 0$

Therefore, $\qquad \frac{d^2\,u}{d\,z^2} = \frac{d^3\,u}{d\,z^3} = 0$

Hence, from Equation (16) and the first of Equations (17), putting $z = 0$

$$\left.\begin{aligned} \frac{w\,R^2\,d}{E_0\,t} + A + C &= 0 \\ -\frac{w\,r^2}{E_0\,t} + \frac{\mu}{\sqrt{2}}\,[(A + B) - (C - D)] &= 0 \end{aligned}\right\} \qquad \ldots\ldots\ldots (22)$$

For the crest of the dam, putting $z = d$ in the second and third of Equations (17) and equating to zero as before

$$\left.\begin{aligned} -A\,e^{\frac{\mu\,d}{\sqrt{2}}} \sin.\frac{\mu\,d}{\sqrt{2}} + B\,e^{\frac{\mu\,d}{\sqrt{2}}} \cos.\frac{\mu\,d}{\sqrt{2}} + C\,e^{-\frac{\mu\,d}{\sqrt{2}}} \sin.\frac{\mu\,d}{\sqrt{2}} \\ - D\,e^{-\frac{\mu\,d}{\sqrt{2}}} \cos.\frac{\mu\,d}{\sqrt{2}} = 0 \\ -A\,e^{\frac{\mu\,d}{\sqrt{2}}}\left(\cos.\frac{\mu\,d}{\sqrt{2}} + \sin.\frac{\mu\,d}{\sqrt{2}}\right) + B\,e^{\frac{\mu\,d}{\sqrt{2}}}\left(\cos.\frac{\mu\,d}{\sqrt{2}} - \sin.\frac{\mu\,d}{\sqrt{2}}\right) \\ + C\,e^{-\frac{\mu\,d}{\sqrt{2}}}\left(\cos.\frac{\mu\,d}{\sqrt{2}} - \sin.\frac{\mu\,d}{\sqrt{2}}\right) \\ + D\,e^{-\frac{\mu\,d}{\sqrt{2}}}\left(\cos.\frac{\mu\,d}{\sqrt{2}} + \sin.\frac{\mu\,d}{\sqrt{2}}\right) = 0 \end{aligned}\right\} \qquad (23)$$

Since

$$\mu^4 = 12\,\frac{E_0}{E_1} \times \frac{1}{t^2\,R^2}$$

$$\frac{\mu}{\sqrt{2}} = \frac{1}{\sqrt{t\,R}}\left(3\,\frac{E_0}{E_1}\right)^{\frac{1}{4}}$$

Suppose, for simplicity, $E_0 = E_1$; then

$$\frac{\mu}{\sqrt{2}} = \frac{3\frac{1}{4}}{\sqrt{t\,R}} = \frac{1.3}{\sqrt{t\,R}}, \text{ nearly.}$$

In most ordinary cases, $e^{-2\frac{\mu\,d}{\sqrt{2}}}$ can be neglected. Dividing the two Equations (23) by $e^{\frac{\mu\,d}{\sqrt{2}}}$ and neglecting $e^{-2\frac{\mu\,d}{\sqrt{2}}}$, there is found

$$-A \sin.\frac{\mu\,d}{\sqrt{2}} + B \cos.\frac{\mu\,d}{\sqrt{2}} = 0$$

$$-A\left(\cos.\frac{\mu\,d}{\sqrt{2}} + \sin.\frac{\mu\,d}{\sqrt{2}}\right) + B\left(\cos.\frac{\mu\,d}{\sqrt{2}} - \sin.\frac{\mu\,d}{\sqrt{2}}\right) = 0$$

So that

$$A = B = 0$$

Hence, from Equation (22)

$$C = -\frac{w\,R^2\,d}{E_0\,t}$$

$$D = \frac{w\,R^2}{E_0\,t}\left(\frac{\sqrt{2}}{\mu} - d\right)$$

The solution of Equation (16) then becomes, using Equation (21),

$$u = \frac{w\,R^2}{E_0\,t}(d - z) - \frac{w\,R^2}{E_0\,t}\,e^{-\frac{\mu\,z}{\sqrt{2}}}\left[d \cos.\frac{\mu\,z}{\sqrt{2}} + \left(d - \frac{\sqrt{2}}{\mu}\right)\sin.\frac{\mu\,z}{\sqrt{2}}\right]$$

In order to fix the meaning of these equations, consider a cylindrical dam 18 ft. in diameter, 20 ft. deep, and 9 in. thick. In this case, taking $E_0 = E_1$,

$$\frac{\mu}{\sqrt{2}} = \frac{1.3}{\sqrt{\frac{3}{4} \times 9}} = \frac{1.3}{2.6} = \frac{1}{2}$$

very nearly and

$$u = \frac{w\,R^2}{E_0\,t}\left[(d - z) - e^{-\frac{1}{2}z}\left\{d \cos.\frac{1}{2}z + (d - z)\sin.\frac{1}{2}z\right\}\right] \quad (24)$$

the exponential term becomes very small when z is greater than 4 ft. above the base, and this portion of the expression therefore may be neglected, thus reducing to

$$u = \frac{w\,R^2}{E_0\,t}(d - z) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

from which, using Equations (4) and (21)

$$T = p\,R.$$

So that the ordinary cylinder formula becomes applicable within quite a moderate distance of the base, and similarly in any case when $\frac{\mu z}{\sqrt{2}}$ becomes large.

The bending moment at the base is given by Equation (3) and the second of Equations (17). Thus, observing that $z = 0$ and using the values of constants just determined,

$$M = C_1 \frac{d^2 u}{dz^2}$$

$$= \frac{1}{12} E_1 t^3 \times \mu^2 (B - D)$$

$$= -\frac{1}{12} E_1 t^3 \times \mu^2 D$$

$$= -\frac{1}{12} E_1 t^3 \times \mu^2 \times \frac{w R^2}{E_0 t} \left(\frac{\sqrt{2}}{\mu} - d \right)$$

$$= -\frac{w}{\mu^2} \left(\frac{\sqrt{2}}{\mu} - d \right)$$

$$= \frac{w}{\mu^2} \left(d - \frac{\sqrt{2}}{\mu} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

In the case considered above, $\mu^2 = \frac{1}{2}$ and $M = 2\,w\,(d - 2) = 2 \times 62.5 \times 18 = 2\,250$ ft-lb. per foot width.

If instead of a dam in which the water pressure is applied outside, the case of a tank is considered, the sign of p is changed. This case may be dealt with by changing the sign of w, which will then change the arch thrust into a tension and change the sign of the bending moment, and also of course change the sign of u.

Case 2.—Arched Dams of Wedge-Shaped Section.

In treating the case of arched dams with wedge-shaped sections in which the thickness is finite at the crest and increases uniformly with the depth, take the origin at the intersection of the up-stream and down-stream faces and measure z vertically downward. Let

$$t = c\,z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

If l_1 is the distance of the water surface below the origin,

$$p = w\,(z - l_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

where w is the pressure, in pounds per square foot per foot in depth, or the weight of 1 cu. ft. of water, 62.5 lb. Then

$$C_1 = E_1 I = \frac{1}{12} E_1 t^3 = \frac{1}{12} E_1 c^3 z^3 \quad \ldots\ldots\ldots\ldots (29)$$

Substituting in Equation (7)

$$\frac{1}{12} E_1 c^3 \frac{d^2}{d z^2}\left(z^3 \frac{d^2 u}{d z^2}\right) + \frac{E_0}{R^2} \times c \times z u = p$$

or dividing by $\frac{1}{12} E_1 c^3$

$$\frac{d^2}{d z^2}\left(z^3 \frac{d^2 u}{d z^2}\right) + 12 \cdot \frac{E_0}{E_1} \times \frac{1}{c^2 R^2} \times z u = \frac{12 p}{E_1 c^3} \quad \ldots\ldots\ldots (30)$$

Let

$$z = a x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Substituting in Equation (30) and multiplying by a

$$\frac{d^2}{d x^2}\left(x^3 \frac{d^2 u}{d x^2}\right) + 12 \frac{E_0}{E_1} \times \frac{a^2}{c^2 R^2} \times x u = \frac{12 p a}{E_1 c^3} \quad \ldots\ldots\ldots (32)$$

If $l_1 = a l'$, the right-hand side of Equation (32) becomes, using Equation (28)

$$\frac{12 w a^2 (x - l')}{E_1 c^3}$$

Now, suppose that a is chosen so that

$$12 \frac{E_0}{E_1} \times \frac{a^2}{c^2 R^2} = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Then Equation (32) becomes

$$\frac{d^2}{d x^2}\left(x^3 \frac{d^2 u}{d x^2}\right) + x u = \frac{w R^2}{E_0 c} (x - l') \quad \ldots\ldots\ldots\ldots (34)$$

which is the fundamental equation on which the solution is based.

A particular integral of Equation (34) is of the form

$$u = A + \frac{B}{x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

for this gives

$$u x = A x + B$$

and

$$\frac{d u}{d x} = -\frac{B}{x^2}$$

$$\frac{d^2 u}{d x^2} = +\frac{2 B}{x^3}$$

so that

$$x^3 \frac{d\,u}{d\,x^2} = 2\,B$$

$$\frac{d}{d\,x}\left(x^3 \frac{d^2\,u}{d\,x^2}\right) = 0$$

$$\frac{d^2}{d\,x^2}\left(x^3 \frac{d^2\,u}{d\,x^2}\right) = 0$$

Substituting in Equation (34)

$$A\,x + B = \frac{w\,R^2}{E_0\,c}\,(x - l')$$

Hence

$$A = \frac{w}{E_0}\frac{R^2}{c}$$

$$B = -\frac{w\,R^2}{E_0\,c}\,l'$$

so that a particular integral of Equation (34) is

$$u = \frac{w\,R^2}{E_0\,c}\left(1 - \frac{l'}{x}\right) \qquad (36)$$

A general solution (the complementary function) of the equation

$$\frac{d^2}{d\,x^2}\left(x^3 \frac{d^2\,u}{d\,x^2}\right) + x\,u = 0 \qquad (37)$$

may be stated, as follows:

Solution in Series—Michell's Functions.

The four solutions of Equation (37) in series form are

$$\left.\begin{aligned}
u_1 &= 1 - \frac{3\,x^2}{(\underline{|3})^2} + \frac{5\,x^4}{(\underline{|5})^2} - \cdots \\
u_2 &= 4\left[\frac{1\,.\,x}{(\underline{|2})^2} - \frac{2\,.\,x^3}{(\underline{|4})^2} + \frac{3\,.\,x^5}{(\underline{|6})^2} - \cdots\right] \\
u_3 &= u_1\,(\log.\,x + 2) + \left[1 - \frac{x^2}{(\underline{|3})^2} + \frac{x^4}{(\underline{|5})^2} - \cdots\right] \\
&\quad - 2\left[1 - \frac{3\,x^2}{(\underline{|3})^2}\,S_3 + \frac{5\,x^4}{(\underline{|5})^2}\,S_5 - \cdots\right] \\
u_4 &= -\frac{1}{2}\,u_2 \log.\,x + \frac{1}{x} - \frac{x}{(\underline{|2})^2} + \frac{x^3}{(\underline{|4})^2} - \cdots \\
&\quad + 4\left[\frac{1\,.\,x}{(\underline{|2})^2}\,S_2 - \frac{2\,.\,x^3}{(\underline{|4})^2}\,S_4 + \cdots\right]
\end{aligned}\right\} \qquad (38)$$

where $$S_n = \frac{1}{1} + \frac{1}{2} + \ldots\ldots + \frac{1}{n}$$

and

$$\underline{|n} = 1 \times 2 \times 3 \times \ldots . n$$

This can be verified by differentiation of each series in Equation (38). These will be called Michell's functions.

Solution in Complex Bessel Functions.

A solution of Equation (37) may also be obtained as follows:

Let

$$\left.\begin{aligned} \zeta^4 + (4x)^2 &= 0 \\ \zeta &= \pm \sqrt{2x}\,(1 \pm i) \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Then a solution of Equation (37) is

$$w = \frac{1}{\zeta} J_1(\zeta) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

where J_1 is a Bessel's function of the first order, defined by Bessel's equation

$$\frac{d^2 J_n}{d\zeta^2} + \frac{1}{\zeta} \cdot \frac{d J_n}{d\zeta} + \left(1 - \frac{n^2}{\zeta^2}\right) J_n = 0 \quad \ldots\ldots\ldots\ldots (41)$$

with $n = 1$. This can also be verified by differentiating Equation (40) and using Equations (39) and (41). The four values of ζ give rise to four different solutions of Equation (37).

Connection Between Michell's and Bessel's Functions.

By comparison of the known series for Bessel's functions with the series given above for Michell's functions,

$$\left.\begin{aligned} w_1 &= \frac{1}{\zeta} J_1(\zeta) + \frac{1}{\zeta'} J_1(\zeta') \\ w_2 &= 2i \left[\frac{1}{\zeta}\,(J_1(\zeta) - \frac{1}{\zeta'} J_1(\zeta'))\right] \\ w_3 &= -2(\log. 2 - 1)\, w_1 - \frac{\pi}{4} w_2 + 2\left[\frac{1}{\zeta} Y_1(\zeta) + \frac{1}{\zeta'} Y_1(\zeta')\right] \\ w_4 &= -\frac{\pi}{2} w_1 + (\log. 2)\, w_2 - 2i\left[\frac{1}{\zeta} Y_1(\zeta) - \frac{1}{\zeta'} Y_1(\zeta')\right] \end{aligned}\right\} \quad (42)$$

where

$$\zeta = \sqrt{2x}\,(1 + i)$$

$$\zeta' = \sqrt{2x}\,(1 - i)$$

and Y_1* is the usual second solution of Bessel's equation for $n = 1$. Equations (38) and (42) are of course convertible one from the other.

The complete solution of Equation (34) is

$$u = \frac{w R^2}{E_0 c}\left(1 - \frac{l'}{x}\right) + A u_1 + B u_2 + C u_3 + D u_4 \ldots\ldots (43)$$

where A, B, C, D are the four arbitrary constants which are to be determined by the boundary conditions at the crest and at the base, exactly as in Case 1.

Tables of Michell's Functions.

In order to facilitate calculation, tables of Michell's functions have been prepared, giving u_1, u_2, u_3, u_4 and their first three derivatives for all integral values of x from 1 to 30. Attention is drawn to the fact that the values of x are not actual heights, but are merely numbers connected with the height by the relation $z = a x$, where a is given by Equation (33), namely,

$$12 \frac{E_0}{E_1} \times \frac{a^2}{c^2 R^2} = 1$$

For example, if

$$E_1 = E_0$$

$$c = \frac{1}{6}$$

$$R = 150$$

Then

$$a^2 = \left(\frac{1}{6} \times 150\right)^2 \times \frac{1}{12}$$

$$a = \frac{25}{2\sqrt{3}} = 7.2 = \frac{36}{5}, \text{ about.}$$

and the values of x are about $\frac{5}{36}$ of the actual vertical height, so that for a dam of these proportions, 180 ft. high, the value of x is 25.

The tables have been calculated directly from Equation (38). For values of x above 10, this work is troublesome on account of the large number of terms which must be included—and for values of x above 20 the tables cannot be relied on for more than about four

* There is an error in Gray and Matthews' expression for Y_n. The sign of the last term $\left(\Sigma'' \frac{1}{2\,8}\right)$ should be $+$, not $-$ as given. The correct expression is given in Jahnke and Emde's Tables.

TABLE 1.—MICHELL'S FUNCTIONS.

x	u_1	$\frac{d\,u_1}{d\,x}$	$\frac{d^2\,u_1}{d\,x^2}$	$\frac{d^3\,u_1}{d\,x^3}$
0.0	+ 1.000 000 00	0.000 000 00	— 0.166 666 67	0.000 000 00
0.5	+ 0.979 188 37	— 0.083 159 77	— 0.165 625 52	+ 0.004 162 54
1.0	+ 0.917 013 61	— 0.165 279 43	— 0.162 508 27	+ 0.008 300 28
1.5	+ 0.814 254 68	— 0.245 325 05	— 0.157 333 48	+ 0.012 388 56
2	+ 0.672 204 62	— 0.322 275 06	— 0.150 132 03	+ 0.016 402 85
3	+ 0.277 924 57	— 0.462 900 59	— 0.129 833 52	+ 0.024 112 71
4	— 0.245 568 72	— 0.579 461 96	— 0.102 100 76	+ 0.031 240 37
5	— 0.870 598 64	— 0.664 846 68	— 0.067 607 44	+ 0.037 604 44
6	— 1.562 742 75	— 0.712 704 78	— 0.027 203 43	+ 0.043 034 32
7	— 2.281 681 64	— 0.717 619 62	+ 0.018 097 18	+ 0.047 372 63
8	— 2.982 211 69	— 0.675 264 69	+ 0.067 130 53	+ 0.050 477 59
9	— 3.615 407 80	— 0.582 546 36	+ 0.118 599 70	+ 0.052 225 29
10	— 4.129 918 35	— 0.437 722 61	+ 0.171 093 76	+ 0.052 511 72
11	— 4.473 374 86	— 0.240 516 84	+ 0.223 108 43	+ 0.051 254 73
12	— 4.593 893 87	+ 0.007 810 04	+ 0.273 068 90	+ 0.048 395 59
13	— 4.441 650 25	+ 0.304 396 08	+ 0.319 354 00	+ 0.043 900 50
14	— 3.970 495 65	+ 0.644 745 53	+ 0.360 321 76	+ 0.037 761 84
15	— 3.139 599 09	+ 1.022 721 39	+ 0.394 335 93	+ 0.029 999 04
16	— 1.915 079 57	+ 1.430 564 76	+ 0.419 793 88	+ 0.020 659 31
17	— 0.271 608 29	+ 1.858 942 21	+ 0.435 153 85	+ 0.009 818 12
18	+ 1.806 054 11	+ 2.297 021 19	+ 0.438 963 88	— 0.002 420 86
19	+ 4.321 605 72	+ 2.732 573 71	+ 0.429 889 85	— 0.015 925 78
20	+ 7.265 874 30	+ 3.152 108 15	+ 0.406 743 22	— 0.030 537 14
21	+ 10.615 627 17	+ 3.541 028 62	+ 0.368 508 53	— 0.046 068 67
22	+ 14.332 564 33	+ 3.883 821 38	+ 0.314 369 58	— 0.062 308 22
23	+ 18.362 491 81	+ 4.164 266 66	+ 0.243 734 64	— 0.079 019 39
24	+ 22.634 750 91	+ 4.365 674 43	+ 0.156 259 75	— 0.095 943 11
25	+ 27.061 858 63	+ 4.471 144 92	+ 0.051 870 58	— 0.112 799 69
26	+ 31.539 446 51	+ 4.463 845 83	— 0.069 218 03	— 0.129 290 98
27	+ 35.946 464 68	+ 4.327 312 54	— 0.206 485 17	— 0.145 103 20
28	+ 40.145 720 43	+ 4.045 759 93	— 0.359 089 25	— 0.159 909 51
29	+ 43.984 701 53	+ 3.604 409 37	— 0.525 856 73	— 0.173 373 28
30	+ 47.296 773 74	+ 2.989 826 06	— 0.705 273 93	— 0.185 151 34

TABLE 1.—MICHELL'S FUNCTIONS (*Continued*).

x	u_2	$\frac{d\,u_2}{d\,x}$	$\frac{d^2\,u_2}{d\,x^2}$	$\frac{d^3\,u_2}{d\,x^3}$
0	0.000 000 00	+ 1.000 000 00	0.000 000 00	− 0.083 333 33
0.5	+ 0.498 264 61	+ 0.989 590 56	− 0.041 608 81	− 0.082 986 24
1.0	+ 0.986 134 25	+ 0.958 449 00	− 0.082 870 78	− 0.081 946 51
1.5	+ 1.453 300 61	+ 0.906 835 16	− 0.123 440 64	− 0.080 218 78
2	+ 1.889 628 37	+ 0.835 180 77	− 0.162 976 19	− 0.077 810 79
3	+ 2.630 603 51	+ 0.634 324 87	− 0.237 600 21	− 0.071 000 18
4	+ 3.134 653 96	+ 0.362 681 67	− 0.304 125 19	− 0.061 637 08
5	+ 3.335 460 91	+ 0.029 600 16	− 0.360 079 53	− 0.049 890 94
6	+ 3.177 260 17	− 0.353 191 40	− 0.403 183 80	− 0.035 976 37
7	+ 2.617 117 84	− 0.771 799 91	− 0.431 394 77	− 0.020 150 68
8	+ 1.626 970 39	− 1.210 424 47	− 0.442 946 81	− 0.002 711 11
9	+ 0.195 386 64	− 1.651 653 30	− 0.436 389 88	+ 0.016 008 77
10	− 1.670 984 70	− 2.076 797 10	− 0.410 624 26	+ 0.035 642 75
11	− 3.946 317 94	− 2.466 254 73	− 0.364 930 94	+ 0.055 796 29
12	− 6.584 893 70	− 2.799 907 55	− 0.298 997 71	+ 0.076 051 10
13	− 9.520 788 54	− 3.057 537 21	− 0.212 940 47	+ 0.095 970 28
14	− 12.667 991 09	− 3.219 262 07	− 0.107 319 11	+ 0.115 103 45
15	− 15.920 965 31	− 3.265 987 15	+ 0.016 851 73	+ 0.132 992 51
16	− 19.155 663 04	− 3.179 861 76	+ 0.158 097 74	+ 0.149 177 26
17	− 22.230 999 12	− 2.944 739 20	+ 0.314 485 97	+ 0.163 201 55
18	− 24.990 784 34	− 2.546 632 30	+ 0.483 631 82	+ 0.174 619 26
19	− 27.266 108 15	1.974 159 47	+ 0.662 712 11	+ 1.183 000 57
20	− 28.878 160 46	1.218 973 04	+ 0.848 484 67	+ 0.187 938 11
21	− 29.641 471 50	− 0.276 167 16	+ 1.037 313 72	+ 0.189 053 12
22	− 29.367 546 82	+ 0.855 344 59	+ 1.225 201 64	+ 0.186 001 69
23	− 27.868 859 01	+ 2.172 484 82	+ 1.407 826 66	+ 0.178 480 55
24	− 24.963 161 96	+ 3.667 711 54	+ 1.580 586 63	+ 0.166 232 86
25	− 20.478 106 07	+ 5.328 760 15	+ 1.738 647 89	+ 0.149 053 82
26	− 14.256 042 31	+ 7.138 439 21	+ 1.877 000 43	+ 0.126 795 76
27	− 6.159 046 69	+ 9.074 483 33	+ 1.990 517 00	+ 0.099 373 07
28	+ 3.915 959 04	+ 11.109 468 21	+ 2.074 017 81	+ 0.066 766 59
29	+ 16.082 060 19	+ 13.210 791 30	+ 2.122 338 41	+ 0.029 027 64
30	+ 30.357 139 10	+ 15.340 724 50	+ 2.130 402 95	− 0.013 718 64

TABLE 1.—MICHELL'S FUNCTIONS (*Continued*).

x	u_3	$\frac{d\,u_3}{d\,x}$	$\frac{d^2\,u_3}{d\,x^2}$	$\frac{d^3\,u_3}{d\,x^3}$
0	$-\infty$	$+\infty$	$-\infty$	$+\infty$
0.5	+ 0.349 004 77	+ 2.126 719 09	— 4.144 436 91	+ 15.658 444 23
1.0	+ 1.110 290 19	+ 1.135 953 82	— 1.035 186 21	+ 1.822 719 02
1.5	+ 1.576 001 25	+ 0.765 643 46	— 0.552 622 35	+ 0.470 733 17
2	+ 1.897 286 64	+ 0.531 027 16	— 0.411 210 41	+ 0.157 286 94
3	+ 2.239 378 55	+ 0.163 228 26	— 0.346 617 64	+ 0.015 074 63
4	+ 2.230 395 63	— 0.181 058 30	— 0.345 095 73	— 0.004 824 19
5	+ 1.876 011 40	— 0.528 346 17	— 0.348 718 95	— 0.000 476 59
6	+ 1.173 657 67	— 0.875 537 05	— 0.343 705 73	+ 0.011 201 12
7	+ 0.128 715 35	— 1.211 303 06	— 0.325 409 68	+ 0.025 675 90
8	— 1.240 371 91	— 1.521 304 00	— 0.291 993 12	+ 0.041 256 34
9	— 2.900 136 83	— 1.790 029 50	— 0.242 819 28	+ 0.057 079 05
10	— 4.861 410 45	— 2.001 704 52	— 0.177 949 01	+ 0.072 563 34
11	— 6.879 371 71	— 2.140 883 82	— 0.097 962 53	+ 0.087 238 49
12	— 9.054 118 78	— 2.192 927 27	— 0.003 882 53	+ 0.100 682 94
13	— 11.231 692 52	— 2.144 423 19	+ 0.102 861 84	+ 0.112 503 48
14	— 13.305 497 46	— 1.983 582 03	+ 0.220 459 24	+ 0.122 329 02
15	— 15.158 119 16	— 1.700 607 20	+ 0.346 738 07	+ 0.129 810 42
16	— 16.663 486 32	— 1.288 044 38	+ 0.479 190 22	+ 0.134 623 04
17	— 17.689 383 31	— 0.741 105 97	+ 0.614 995 85	+ 0.136 470 94
18	— 18.100 261 75	— 0.057 965 33	+ 0.751 055 62	+ 0.135 090 38
19	— 17.760 342 35	+ 0.759 977 28	+ 0.884 024 64	+ 0.130 253 90
20	— 16.536 949 62	+ 1.707 871 62	+ 1.010 351 55	+ 0.121 779 09
21	— 14.304 071 56	+ 2.777 229 12	+ 1.126 322 11	+ 0.109 522 89
22	— 10.946 049 13	+ 3.955 787 69	+ 1.228 106 47	+ 0.093 395 61
23	— 6.361 430 31	+ 5.227 415 57	+ 1.311 809 78	+ 0.073 358 90
24	— 0.466 832 30	+ 6.572 076 93	+ 1.373 526 39	+ 0.049 430 10
25	+ 6.799 134 31	+ 7.965 851 35	+ 1.409 397 20	+ 0.021 684 77
26	+ 15.472 035 19	+ 9.381 002 59	+ 1.415 668 81	— 0.009 740 74
27	+ 25.557 832 98	+ 10.786 124 32	+ 1.388 754 59	— 0.044 648 47
28	+ 37.029 338 23	+ 12.146 327 15	+ 1.325 297 95	— 0.082 776 97
29	+ 49.822 839 52	+ 13.423 513 56	+ 1.222 235 05	— 0.123 801 40
30	+ 63.835 054 69	+ 14.576 687 67	+ 1.076 858 96	— 0.167 333 26

TABLE 1.—MICHELL'S FUNCTIONS (*Continued*).

x	u_4	$\frac{d\,u_4}{d\,x}$	$\frac{d^2\,u_4}{d\,x^2}$	$\frac{d^3 u_4}{d\,x^3}$
0	+ ∞	− ∞	+ ∞	− ∞
0.5	+ 2.794 287 19	2.925 680 85	+ 14.921 467 33	− 94.114 880 95
1.0	+ 2 222 855 68	− 0.324 390 60	+ 1.372 518 26	− 5.584 067 47
1.5	+ 2.155 654 02	− 0.044 932 10	+ 0.094 914 87	1.027 358 73
2	+ 2.129 261 86	− 0.083 876 56	− 0.192 562 13	− 0.298 945 42
3	+ 1.917 207 03	0.360 304 18	− 0.320 973 68	− 0.042 036 12
4	+ 1.392 154 36	− 0.691 902 35	− 0.334 432 07	+ 0.006 357 67
5	+ 0.535 173 75	− 1.018 939 64	− 0.315 865 84	+ 0.029 250 11
6	− 0.636 048 21	− 1.317 101 62	− 0.277 489 98	+ 0.047 099 60
7	− 2.083 355 94	− 1.568 297 44	− 0.222 211 95	+ 0.063 242 81
8	− 3.751 582 58	− 1.756 353 31	− 0.151 418 05	+ 0.078 128 62
9	− 5.570 043 42	− 1.866 399 35	− 0.066 431 09	+ 0.091 581 37
10	7.453 883 62	− 1.885 019 29	+ 0.031 159 51	− 0.103 278 11
11	− 9.305 673 20	− 1.800 518 82	+ 0.139 423 62	+ 0.112 870 86
12	− 11.017 337 84	− 1.603 359 64	+ 0.256 087 58	− 0.120 023 73
13	− 12.472 430 03	− 1.286 405 96	+ 0.378 554 21	+ 0.124 427 11
14	− 13.548 725 43	− 0.845 277 40	+ 0.503 933 06	− 0.125 805 31
15	− 14.121 105 43	− 0.278 625 73	+ 0.629 076 96	+ 0.123 921 27
16	− 14.064 708 02	+ 0.411 679 21	+ 0.750 623 21	+ 0.118 581 57
17	− 13.258 279 78	+ 1.220 255 18	+ 0.865 088 76	+ 0.109 639 66
18	− 11.587 713 13	+ 2.138 162 36	+ 0.968 669 32	+ 0.097 060 02
19	− 8.949 685 37	+ 3.152 758 15	+ 1.057 791 69	+ 0.080 620 83
20	− 5.255 386 92	+ 4.247 663 57	+ 1.128 668 90	+ 0.060 516 98
21	− 0.434 246 91	+ 5.402 781 86	+ 1.177 608 08	+ 0.036 762 19
22	+ 5.562 370 74	+ 6.594 369 44	+ 1.201 020 39	+ 0.009 490 77
23	+ 12.757 600 27	+ 7.795 169 95	+ 1.195 482 01	− 0.021 101 15
24	+ 21.145 624 79	+ 8.974 613 93	+ 1.157 796 86	0.054 754 82
25	+ 30.688 529 39	+ 10.099 074 80	+ 1.085 058 24	0.091 149 31
26	+ 41.313 351 99	+ 11.132 189 61	+ 0.974 711 76	− 0.129 901 64
27	+ 52.909 577 44	+ 12.035 243 78	+ 0.824 616 45	− 0.170 568 24
28	+ 65.343 611 83	+ 12.767 789 77	+ 0.633 097 76	− 0.212 646 48
29	+ 78.373 901 86	+ 13.287 260 53	+ 0.399 037 69	− 0.255 579 30
30	+ 91.816 273 29	+ 13.551 308 17	+ 0.121 864 48	− 0.298 753 96

decimal places. In case it should be desired to use the functions for higher values than those given in the tables, it has been thought advisable to give the semi-converging expressions for Michell's functions, as follows. Let*

$$P_n = 1 - \frac{(4n^2-1^2)(4n^2-3^2)}{\underline{|2}\,(8\zeta)^2} + \frac{(4n^2-1^2)(4n^2-3^2)(4n^2-5^2)(4n^2-7^2)}{\underline{|4}\,(8\zeta)^4} - \ldots.$$

$$Q_n = \frac{4n^2-1^2}{8\zeta} - \frac{(4n^2-1^2)(4n^2-3^2)(4n^2-5^2)}{\underline{|3}\,(8\zeta)^3} + \ldots.$$

and putting $n = 1$, there results

$$P_1 = 1 + \frac{3\times 5}{\underline{|2}\,(8\zeta)^2} - \frac{3\times 5\times 21\times 45}{\underline{|4}\,(8\zeta)^4} + \ldots.$$

$$Q_1 = \frac{3}{8\zeta} - \frac{3\times 5\times 21}{\underline{|3}\,(8\zeta)^3} + \ldots.$$

write

$$\zeta = \sqrt{2x}\,(1+i)$$
$$\zeta' = \sqrt{2x}\,(1-i)$$
$$P_1 = A - iB$$
$$Q_1 = C - iD$$
$$P_1^1 = A + iB$$
$$Q_1^1 = C + iD$$

where

$$A = 1 + 0.009012 \times \frac{1}{x^2}$$

$$B = +0.029297 \times \frac{1}{x}$$

$$C = \frac{1}{16\sqrt{2x}}\left(3 + 0.20508 \times \frac{1}{x}\right)$$

$$D = \frac{1}{16\sqrt{2x}}\left(3 - 0.20508 \times \frac{1}{x}\right)$$

and call

$$\alpha = \left(\sin.\sqrt{2x} - \cos.\sqrt{2x}\right)\cosh.\sqrt{2x}$$
$$\beta = \left(\sin.\sqrt{2x} + \cos.\sqrt{2x}\right)\sinh.\sqrt{2x}$$
$$\gamma = \left(\sin.\sqrt{2x} + \cos.\sqrt{2x}\right)\cosh.\sqrt{2x}$$
$$\delta = \left(\sin.\sqrt{2x} - \cos.\sqrt{2x}\right)\sinh.\sqrt{2x}$$

* Gray and Matthews' "Treatise on Bessel Functions", p. 38 (MacMillan and Co., London and New York).

where, as usual, the hyperbolic functions are defined by

$$\cosh.\ \theta = \frac{1}{2}\,(e^{\theta} + e^{-\theta})$$

$$\sinh.\ \theta = \frac{1}{2}\,(e^{\theta} - e^{-\theta})$$

also let

$$H = \sqrt{\sqrt{2}+1} - \sqrt{\sqrt{2}-1} = 0.910181$$

$$K = \sqrt{\sqrt{2}+1} + \sqrt{\sqrt{2}-1} = 2.197365$$

Then the semi-convergent expressions for Michell's functions* are from the expressions for J_n and Υ_n in Gray and Matthews, by substituting $\zeta = \sqrt{2x}\,(1+i)$ and $\zeta' = \sqrt{2x}\,(1-i)$ in

$$J_n(\xi)\sqrt{\frac{2}{\pi\xi}}\left[P_n \cos.\left(\frac{(2n+1)\,\pi}{4} - \xi\right) + Q_n \sin.\left(\frac{(2n+1)\,\pi}{4} - \xi\right)\right]$$

$$\Upsilon_n(\xi) = (\log.\,2 - \gamma)\,J_n(\xi) + \sqrt{\frac{\pi}{2\xi}}\left[Q_n \cos.\left(\frac{(2n+1)\,\pi}{4} - \xi\right) - P_n \sin.\left(\frac{(2n+1)\,\pi}{4} - \xi\right)\right]$$

where P_n and Q_n have the values given above and γ is Euler's constant 0.577215.....while log. 2 is the Naperian logarithm of 2 or 0.693147 ...., so that $(\log.\,2 - \gamma) = 0.115931....$

The required expressions are

$$u_1 = \frac{1}{2\sqrt{\pi}\,(2x)^{\frac{3}{4}}}\,[H\,(A\,\alpha + B\,\beta + C\,\gamma - D\,\delta) + K\,(A\,\beta - B\,\alpha - C\,\delta - D\,\gamma)]$$

$$u_2 = \frac{1}{\sqrt{\pi}\,(2x)^{\frac{3}{4}}}\,[H\,(-A\,\beta + B\,\alpha + C\,\delta + D\,\gamma) + K\,(A\,\alpha + B\,\beta + C\,\gamma - D\,\delta)]$$

$$u_3 = 2\,(1-\gamma)\,u_1 - \frac{\pi}{4}\,u_2 + \frac{\sqrt{\pi}}{2\,(2x)^{\frac{3}{4}}}\,[H\,(-A\,\gamma + B\,\delta + C\,\alpha + D\,\beta) + K\,(A\,\delta + B\,\gamma + C\,\beta - D\,\alpha)]$$

$$u_4 = -\frac{\pi}{2}\,u_1 + \gamma\,u_2 - \frac{\sqrt{\pi}}{2\,(2x)^{\frac{3}{4}}}\,[H\,(A\,\delta + B\,\gamma + C\,\beta - D\,\alpha) + K\,(A\,\gamma - B\,\delta - C\,\alpha - D\,\beta)]$$

* Gray and Matthews' "Bessel Functions", p. 40.

Connection Between Solution for Dams for Uniform Thickness and Triangular Sections.

In order to establish the connection between the solution in the case of the wedge-shaped section and the dam of constant thickness, suppose z, and therefore also x, to become indefinitely large and c indefinitely small, so that

$$t = c\,z = c\,(z_0 + z')$$

Where z' is very small, $t = c\,z_0$, approximately constant.

Then, referring to Equations (9), (10), and (33),

$$\mu^4 = \frac{E_0\,t}{C_1 R^2} = 12\,\frac{E_0}{E_1}\cdot\frac{1}{t^2\,R^2} = \frac{c^2}{a^2\,t^2} = \frac{1}{a^2\,z_0^2}$$

$$x = \frac{z}{a} = \frac{1}{a}\,(z_0 + z') = \frac{z_0}{a}\left(1 + \frac{z'}{z_0}\right)$$

hence

$$\sqrt{2\,x} = \sqrt{\frac{2\,z_0}{a}}\left(1 + \frac{z'}{z_0}\right)^{\frac{1}{2}} = \sqrt{\frac{2\,z_0}{a}\left(1 + \frac{z'}{2\,z_0}\right)} = \sqrt{\frac{2\,z_0}{a}} + \frac{z'}{\sqrt{2\,z_0\,a}}$$

$$= \frac{\sqrt{2}}{a\,\mu} + \frac{\mu\,z'}{\sqrt{2}}$$

in which $\dfrac{\sqrt{2}}{a\,\mu}$ is a constant, so that $\alpha, \beta, \gamma, \delta$ reduce to terms of the form

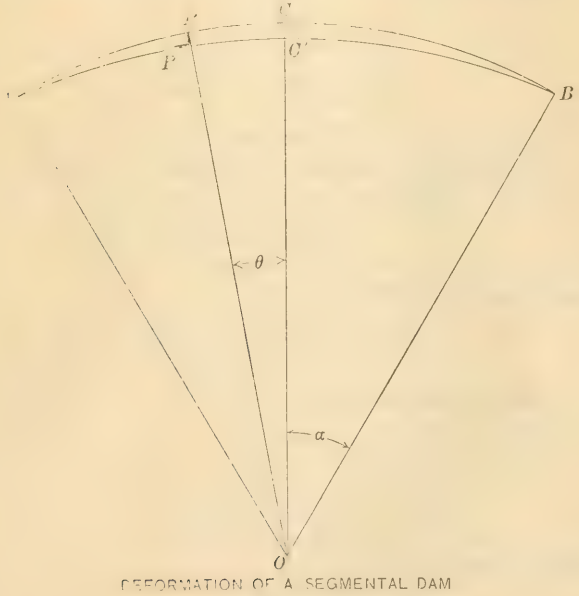
$$\cos\frac{\mu\,z'}{\sqrt{2}}\cdot\cosh.\frac{\mu\,z'}{\sqrt{2}}\ \text{etc.}$$

that is, when x is increased indefinitely and c decreased indefinitely, so as to keep the thickness of the dam constant, Michell's functions reduce to products of circular and hyperbolic functions and the solution degenerates into the form of Equation (16).

APPENDIX II.

DETERMINATION OF THE ELASTIC MODULUS FOR SEGMENTAL DAMS IN TERMS OF YOUNG'S MODULUS E_0 FOR THE MATERIAL USED IN THE DAMS.

Let $A\ C\ B$, Fig. 5, be a segment of a cylinder of unit height and of radius R, subject to a uniform compression S per unit area, free from bending moment and held so that the abutments A and B are fixed in position while the segment can slide freely on its base.



DEFORMATION OF A SEGMENTAL DAM

FIG. 5.

If E_0 is Young's Modulus for the material and f the elastic compression, that is, the ratio of the contraction in length to the original length of any part, then

$$S = E_0 f \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Then, since S is constant throughout, f is constant. If, after deformation, the segment takes up the position $A\ C'\ B$ so that the point P goes to P' and C to C', if

$u =$ the normal displacement toward the axis O

$v =$ the tangential displacement in the direction θ increases, where θ is the angle $P\ O\ C$, C being the middle point of the segment,

then
$$f = \frac{1}{R}\left(-\frac{dv}{d\theta} + u\right) \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

Now by hypothesis f is constant, hence from Equation (45)
$$-\frac{dv}{d\theta} + u = fR = \text{constant} \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

Assume
$$u = u_0 - u_1 \cos.\ \theta \ldots\ldots\ldots\ldots\ldots\ldots (47)$$
which corresponds to a deformation symmetrical about $O\ C$. From Equations (46) and (47)
$$-v = \int_0^\theta (Rf - u)\, d\theta = \int_0^\theta [(Rf - u_0) - u_1 \cos.\ \theta]\, d\theta$$
$$= (Rf - u_0)\,\theta - u_1 \sin.\ \theta \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

When $\theta = 0$, $u = CC' = U$, say; and when $\theta = \pm\alpha$, $u = v = 0$ since A and B are fixed points. Hence, from Equations (47) and (48)
$$\left.\begin{aligned}(Rf - u_0)\,\alpha - u_1 \sin.\ \alpha &= 0\\ u_0 + u_1 \cos.\ \alpha &= 0\\ u_0 + u_1 &= U\end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

Solving Equation (49) for v
$$U = \frac{Rf\,\alpha(1 - \cos.\ \alpha)}{\sin.\ \alpha - \alpha \cos.\ \alpha} \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

When α is small, approximately
$$\sin.\ \alpha = \alpha - \frac{\alpha^3}{\lfloor 3} = \alpha - \frac{\alpha^3}{6}$$
$$\cos.\ \alpha = 1 - \frac{\alpha^2}{\lfloor 2} = 1 - \frac{\alpha^2}{2}$$

Substituting in Equation (50)
$$U = \frac{Rf\,\alpha \times \frac{\alpha^2}{2}}{\frac{\alpha^3}{3}} = \frac{3}{2} Rf \ldots\ldots\ldots\ldots\ldots\ldots (51)$$

which is independent of α so long as α is small. When $\alpha = \frac{\pi}{3}$ (so that the angle $A\ O\ B$ is 120°) the coefficient of Rf is 1.528.... instead of 1.5, so that we may regard Equation (51) as practically true for all ordinary arched dams.

In the given case
$$f = \frac{S}{E_0}$$

from Equation (44), hence,

$$U = \frac{3}{2} \cdot \frac{R}{E_0} S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

Now, if a complete ring of radius R and center O and of Young's Modulus E_0 be subjected to the same stress as above, then

$v = 0$ throughout, and consequently

$$f = \frac{u}{R}$$

Hence,

$$S = E_0 \times f = E_0 \times \frac{u}{R}$$

so that

$$u = \frac{R}{E_0} S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

If S_1, E_0, and R be the same for the segment as for the complete ring, then the displacement of the middle point of the segment is 50% greater than the displacement of any point on the complete ring; that is to say, the displacement of the middle point of the segment is the same as it would be if the segment formed part of a complete cylinder of which the Young's Modulus had a value two-thirds of that which it actually has in the material.

If we suppose the dam divided up into horizontal layers in which S varies from layer to layer, and if we suppose that these layers are free to slide over one another; that is, if we neglect the twist of the surface due to the base being fixed and the displacements not being constant at all points of the arch layers, the displacement of the middle point C will be as given above, and we can obtain the solution for the segmental dam by using the Modulus $\frac{2}{3} E_0$ instead of E_0 in the solution for the complete cylinder.

APPENDIX III.

NUMERICAL EXAMPLE OF CALCULATION OF STRESSES IN A DAM OF TRIANGULAR SECTION.

In the case of the Wooling Dam, the leading dimensions, Fig. 6, are as follows:

Crest width, $A\ C = 2.20$ ft.
Base, $B\ D = 4.40$ ft.
Height, $A\ B = 33$ ft.

So that

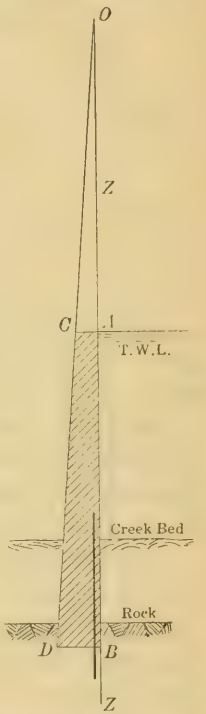
$$c = \frac{2.2}{33} = \frac{1}{15}$$

The radius of the arch to the up-stream face is 135 ft. The angle of the sector is about 120° at crest level, so that E_0 is replaced by $\frac{2}{3}\ E$ throughout. The value adopted for E_1 was 2 600 000 lb. per sq. in. and that for E was 2 000 000 lb. per sq. in., the larger value being adopted for the vertical section to make a rough allowance for the vertical reinforcement and the value $\frac{2}{3}\ E$ being used to allow for the segmental form. Then

$$a = c\ R \sqrt{\frac{1}{12} \cdot \frac{E_1}{\frac{2}{3} E}} = \frac{11}{3} \text{ nearly}$$

$$O\ A = l_1 = 15 \times 2.2 = 33 \text{ ft.}$$
$$O\ B = l_2 = 15 \times 4.4 = 66 \text{ ft.}$$

$$x = z \times \frac{1}{a} = \frac{3}{11} \times z$$



SECTION OF DAM OF TRIANGULAR SECTION, BASE ENCASTRÉE
FIG. 6.

Hence, corresponding to A, $x = l_1' = 9$
and corresponding to B, $x = l_2' = 18$

Remembering that, for the segmental dam, $E_0 = \frac{2}{3}\ E$, and calling

$$u = u' \frac{w\ R^2}{E_0\ c} = u' \frac{w\ R^2}{\frac{2}{3} E\ c}$$

$$A = A' \frac{w\ R^2}{E_0\ c}, \text{ etc.}$$

Equation (43) may be written

$$u' = A' u_1 + B' u_2 + C' u_3 + D' u_4 + 1 - \frac{l'}{x} \ldots\ldots (54)$$

From which, by differentiation

$$\frac{d u'}{d x} = A' \frac{d u_1}{d x} + B' \frac{d u_2}{d x} + C' \frac{d u_3}{d x} + D' \frac{d u_4}{d x} + \frac{l'}{x^2} \ldots. (55)$$

$$\frac{d^2 u'}{d x^2} = A' \frac{d^2 u_1}{d x^2} + B' \frac{d^2 u_2}{d x^2} + C' \frac{d^2 u_3}{d x^2} + D' \frac{d^2 u_4}{d x^2} - \frac{2 l'}{x^3} \ldots. (56)$$

$$\frac{d^3 u'}{d x^3} = A' \frac{d^3 u_1}{d x^3} + B' \frac{d^3 u_2}{d x^3} + C' \frac{d^3 u_3}{d x^3} + D' \frac{d^3 u_4}{d x^3} + \frac{6 l'}{x^4} \ldots. (57)$$

CASE 1.—DAM FULL TO CREST LEVEL—NO TEMPERATURE EFFECT—BOTTOM ENCASTRÉE—CREST FREE.

The end conditions for this case are, observing that since the water is at crest level, $l' = l_1'$

at $x = l_1' = 9$

$$M = F = 0 \qquad \frac{d^2 u'}{d x^2} = \frac{d^3 u'}{d x^3} = 0$$

at $x = l_2' = 18$

$$u' = \frac{d u'}{d x} = 0$$

Equations (54) to (57) thus give

at $x = l_1' = 9$

$$A' \frac{d^2 u_1}{d x^2} + B' \frac{d^2 u_2}{d x^2} + C' \frac{d^2 u_3}{d x^2} + D' \frac{d^2 u_4}{d x^2} = -\frac{2 l_1'}{x^3}$$

$$= \frac{2 \times 9}{9^3} = + 0.024\,691 \ldots.$$

$$A' \frac{d^3 u_1}{d x^3} + B' \frac{d^3 u_2}{d x^3} + C' \frac{d^3 u_3}{d x^3} + D' \frac{d^3 u_4}{d x^3} = -\frac{6 l_1'}{x^4}$$

$$= -\frac{6 \times 9}{9^4} = - 0.008\,230 \ldots.$$

at $x = l_2' = 18$

$$A' u_1 + B' u_2 + C' u_3 + D' u_4 = -1 + \frac{l'}{x} = -1 + \frac{9}{18} = - 0.500\,000 \ldots.$$

$$A' \frac{d u_1}{d x} + B' \frac{d u_2}{d x} + C' \frac{d u_3}{d x} + D' \frac{d u_4}{d x} = -\frac{l'}{x^2}$$

$$= -\frac{9}{18^2} = - 0.027\,777 \ldots.$$

TABLE 2.—*Continued.*

A'	B'	C'	D'	Term on Right	Sum of Coefficients + Term on Right
Hence, eliminating D',					
+ 0.000 534 837	.....	+ 0.000 089 937	.....	+ 0.032 473 402	+ 0.033 098 181
− 0.000 195 930	.....	+ 0.000 184 456	.....	− 0.009 198 353	− 0.009 149 823
From the second of these:					
− 1.0	.....	+ 0.941 438 268	.....	− 46.640 908 998	− 46.699 470 730
Multiplying each term by the coefficient of A' in the first of these:					
− 0.000 534 837	.....	+ 0.000 503 516	.....	− 0.024 945 284	− 0.024 976 605
Eliminating A'					
.....	.....	+ 0.000 593 453	.....	+ 0.007 528 118	+ 0.008 121 576
.....	.....	1.0	.....	+ 12.685 281 73	+ 13.685 281 73

Hence

$C' = +$ 12.685 281 73		+ 0.020 007 38
	+ 0.006 982 15	+ 4.233 746 43
+ 46.640 909 00	− 37.296 157 45	− 9.187 663 73
+ 11.942 409 66	− 6.828 083 08	+ 20.456 266 36
$A' =$ + 58.583 319 66	$D' =$ − 44.117 258 38	$B' =$ + 15.522 356 44

I	II	III	IV
+ 6.947 965	+ 3.059 530	+ 105.804 644	+ 134.567 12
− 6.773 796	+ 0.218 494	− 387.915 862	− 39.529 73
− 3.080 232	+ 0.724 064	− 229.606 919	− 0.735 30
+ 2.930 757	− 4.040 308	+ 511.218 134	− 94.329 87
+ 0.024 694	− 0.008 230	− 0.500 003	− 0.027 78

TABLE 2.—33-FOOT DAM, BASE ENCASTRÉE, CREST FREE—NUMERICAL SOLUTION OF EQUATION (43), TRANSFORMED AS IN EQUATION (54).

The Last Column is the Algebraic Sum of the Terms in the Other Columns and is Used for Checking Each Step.

A'	B'	C'	D'	Term on Right	Sum of Coefficients + Term on Right
$x = 9$					
+ 0.118 599 700	— 0.436 389 880	— 0.242 819 280	— 0.066 431 090	+ 0.024 691 358	— 0.602 349 192
+ 0.052 225 290	+ 0.016 008 770	+ 0.057 079 050	+ 0.091 581 370	— 0.008 230 453	+ 0.208 664 027
$x = 18$					
+ 1.806 054 110	— 24.990 784 340	— 18.100 261 750	— 11.587 713 130	— 0.500 000 000	— 53.372 705 110
+ 2.207 021 190	— 2.546 632 300	— 0.057 965 330	+ 2.138 162 360	— 0.027 777 778	+ 1.802 808 142
From the third of these:					
+ 0.072 268 805	— 1.0	— 0.724 277 468	— 0.463 679 456	— 0.020 007 375	2.135 695 477
Multiplying each of these by the coefficient of B' in Equations (1), (2), and (4)					
+ 0.031 537 375	— 0.436 389 880	— 0.316 067 357	0.202 345 022	— 0.008 731 016	— 0.931 995 903
+ 0.001 156 935	— 0.016 008 770	— 0.011 594 791	— 0.007 422 938	0.000 320 293	— 0.034 189 858
+ 0.184 042 073	— 2.546 632 300	— 1.844 468 394	1.180 821 080	0.050 951 428	— 5.438 831 128
Hence, eliminating B',					
+ 0.087 062 325	.....	+ 0.073 248 077	+ 0.135 913 932	+ 0.033 422 374	+ 0.329 646 713
+ 0.053 382 225	.....	+ 0.045 484 259	+ 0.084 158 432	+ 0.008 550 746	+ 0.174 474 174
+ 2.112 979 117	.....	+ 1.786 503 064	+ 3.318 983 440	+ 0.023 173 651	+ 7.241 639 270
From the third of these:					
+ 0.636 634 426	.....	+ 0.538 268 146	1.0	+ 0.006 982 153	+ 2.181 884 726
Multiplying each of these numbers by the coefficient of D' in the first two:					
+ 0.086 527 488	.....	+ 0.073 158 140	+ 0.135 913 932	+ 0.000 948 972	+ 0.296 548 532
+ 0.053 578 155	....	+ 0.045 299 803	+ 0.084 158 432	+ 0.000 587 607	+ 0.183 623 997

TABLE 3.—(*Continued*).

To Find $\frac{d^2 u'}{d x^2}$

Values of x, in Feet.

	9	10	11	12	13	14	15	16	17	18
$A' \frac{d^2 u_1}{d x^2}$	+ 6.947 965	+ 10.023 241	+ 13.070 437	+ 15.997 282	+ 18.708 875	+ 21.108 844	+ 23.101 505	+ 24.592 915	+ 25.492 754	+ 25.715 959
$B' \frac{d^2 u_2}{d x^2}$	— 6.773 796	— 6.373 854	— 5.664 587	— 4.641 148	— 3.305 337	— 1.665 845	+ 0.261 579	+ 2.454 049	+ 4.881 562	+ 7.507 104
$C' \frac{d^2 u_3}{d x^2}$	— 3.080 232	— 2.257 334	— 1.242 682	— 0.049 251	+ 1.304 832	+ 2.796 588	+ 4.398 471	+ 6.078 663	+ 7.801 396	+ 9.527 356
$D' \frac{d^2 u_4}{d x^2}$	+ 2.930 757	— 1.374 672	— 6.150 987	— 11.297 883	— 16.700 773	— 22.232 148	— 27.753 155	— 33.115 435	— 38.163 140	— 42.735 040
$- \frac{2 l'}{x^3}$	— 0.024 691	— 0.018 000	— 0.013 523	— 0.010 416	— 0.008 193	— 0.006 560	— 0.005 261	— 0.004 394	— 0.003 663	— 0.003 086
	+ 9.878 722	+ 10.023 241	+ 13.070 437	+ 15.997 282	+ 20.013 707	+ 23.905 432	+ 27.761 555	+ 33.125 627	+ 38.175 712	+ 42.750 419
	— 9.878 719	— 10.023 860	— 13.071 779	— 15.998 698	— 20.014 303	— 23.904 553	— 27.758 416	— 33.119 829	— 38.166 803	— 42.738 126
$\frac{d^2 u'}{d x^2}$	+ 0.000 003	— 0.000 619	— 0.001 342	— 0.001 416	— 0.000 596	+ 0.000 879	+ 0.003 139	+ 0.005 798	+ 0.008 909	+ 0.012 293

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TABLE 3.—33-FOOT DAM, BASE ENCASTRÉE, SEGMENTAL—DAM JUST FULL TO CREST LEVEL, NO OVERFLOW.

$$c = \frac{1}{15} \qquad A' = + 58.583\ 319$$

$$a = \frac{11}{3} \qquad B' = + 15.522\ 356$$

$R = 135$ ft. $\qquad C' = + 12.685\ 282$

$$D' = - 44.117\ 258$$

$$u' = A'\,u_1 + B'\,u_2 + C'\,u_3 + D'\,u_4 + \left(1 - \frac{l_1'}{x}\right)$$

$$u \div \frac{w\,R^2}{E_0\,c} = \frac{u}{0.089}$$

TO FIND u'

VALUES OF x, IN FEET.

	9	10	11	12	13	14	15	16	17	18
$A'\ u_1$	− 211.802 57	− 241.944 22	− 262.065 18	− 269.125 80	− 260.206 59	− 232.604 87	− 183.928 18	− 112.191 75	− 15.911 72	+ 105.804 64
$B'\ u_2$	+ 3.032 86	− 25.937 62	− 61.256 17	− 102.214 08	− 147.785 10	− 196.637 10	− 247.130 96	− 297.341 09	− 345.077 54	− 387 915 86
$C'\ u_3$	− 36.789 06	− 60.907 250	− 87.266 76	− 114.854 06	− 142.477 24	− 168.783 99	− 192.285 09	− 211.381 02	− 224.894 84	− 229.606 91
$D'\ u_4$	+ 245.735 03	+ 328.844 84	+ 410.540 75	+ 486.054 72	+ 550.249 49	+ 597.732 46	+ 622.984 57	+ 620.496 43	+ 584.919 05	+ 511.218 13
$1 - \frac{l'}{x^4}$	0.	+ 0.100 000	+ 0.181 82	+ 0.250 00	+ 0.307 70	+ 0.357 15	+ 0.400 00	+ 0.437 50	+ 0.470 59	+ 0.500 00
	+ 248.767 89	+ 328.944 84	+ 410.722 57	+ 486.304 72	+ 550.557 19	+ 598.089 61	+ 623.384 57	+ 620.933 93	+ 585 389 64	+ 617.522 77
	248.591 63	− 328.789 09	− 410.588 11	− 486.193 94	− 550.468 93	− 598.025 96	− 623.344 23	− 620.913 86	− 585.384 10	− 617.522 77
u'	+ 0.176 26	+ 0.155 75	+ 0.134 46	+ 0.110 78	+ 0.088 26	+ 0.063 65	+ 0.040 34	+ 0.020 07	+ 0.005 54	+ 0.000 00

Substituting from the tables and solving according to the tabular form, Table 2:

$$A' = +\,58.583\,319$$
$$B' = +\,15.522\,356$$
$$C' = +\,12.685\,282$$
$$D' = -\,44.117\,258.$$

Observing that

$$M = C_1 \frac{d^2 u}{d z^2} = w a^3 x^3 \frac{d^2 u'}{d x^2}$$

where $a = \frac{11}{3}$ nearly, the bending moments at $x = 9, 10 \ldots\ldots 18$ can be found. Again, observing that by Equation (48)

$$T = \frac{2}{3} E \times \frac{t}{R} \times u = \frac{2}{3} E \times \frac{c z}{R} \times u' \times \frac{w \frac{R^3}{2}}{\frac{2}{3} E c} = w R^2 \times z u'$$

the arch thrust at $x = 9, 10 \ldots\ldots 18$ can be found. Diagrams for the thrust and bending moment are given in Figs. 8 and 9.

TABLE 4.—33-Foot Dam, Base Encastrée, Crest Free—Displacements (u), Thrusts (T), and Bending Moments (M).

$u = 0.089\, u'$

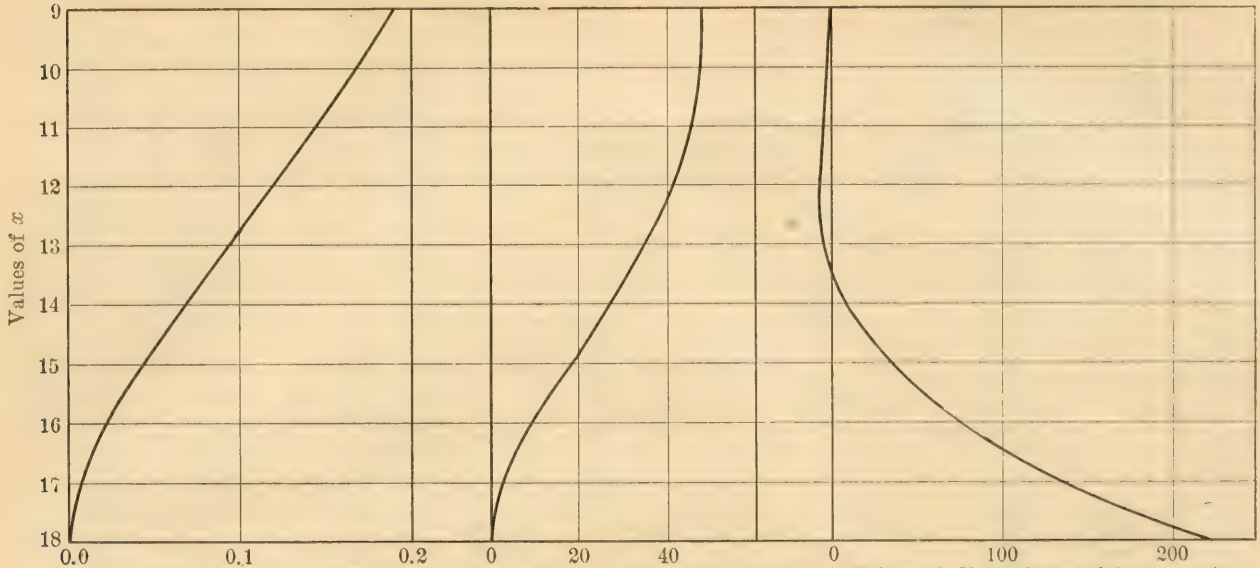
$T = w\, a\, R\, x\, u' = 31\,000\, x\, u'$

$M = w\, a^3\, x^3 \frac{d^2 u'}{d x^2} = 3080\, x^3 \frac{d^2 u'}{d x^2}$

$E_0 = \frac{2}{3} E$, where E is Young's Modulus for concrete

d = depth below crest level, in feet

x	z, in feet.	d, in feet.	u, in inches.	T, in pounds per foot height.	M, in foot-pounds per foot width.
9	33.0	0	0.188	49 000	0
10	36.6	3.6	0.166	48 200	— 1 900
11	40.3	7.3	0.143	45 800	— 5 500
12	44.0	11.0	0.118	41 100	— 7 500
13	47.6	14.6	0.093	35 500	— 4 000
14	51.3	18.3	0.068	27 600	+ 7 400
15	55.0	22.0	0.042	18 700	+ 32 600
16	58.6	25.6	0.021	9 900	+ 73 200
17	62.3	29.3	0.006	2 900	+134 800
18	66.0	33.0	0.000	0	+220 900



Values of u, in inches.

FIG. 7.—DIAGRAM SHOWING DISPLACEMENTS,

Values of T, in thousand pounds.

FIG. 8.—DIAGRAM SHOWING ARCH THRUST, T

Values of M, in thousand foot-pounds.

FIG. 9.—DIAGRAM SHOWING BENDING MOMENTS M

FIGS. 7, 8, 9.—DISPLACEMENT, THRUST AND BENDING MOMENT, 33-FOOT DAM, BASE ENCASTRÉE, CREST FREE.

APPENDIX IV.

NUMERICAL EXAMPLE OF CALCULATION OF TEMPERATURE STRESSES, BASE ENCASTREE, CREST FREE.

To deal with this case, suppose the dam empty and proceed as in Section 6, under the heading "Proposed Method of Design, Using Michell's Functions". Put $w = o$ in Equation (43), which then becomes simply

$$u = A\,u_1 + B\,u_2 + C\,u_3 + D\,u_4 \ldots\ldots\ldots\ldots\ldots (58)$$

Let

$\theta =$ the anticipated rise of temperature above the mean temperature during the construction of the dam;

$e =$ the coefficient of expansion of the concrete

The displacement at the center is v toward the axis of the cylinder, where

$$v = e\,R\,\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

Equations (54) to (57) are now replaced, when $x = l_1 = 9$, by

$$A\,\frac{d^2 u_1}{dx^2} + B\,\frac{d^2 u_2}{dx^2} + C\,\frac{d^2 u_3}{dx^2} + D\,\frac{d^2 u_4}{dx^2} = 0$$

$$A\,\frac{d^3 u_1}{dx^3} + B\,\frac{d^3 u_2}{dx^3} + C\,\frac{d^3 u_3}{dx^3} + D\,\frac{d^3 u_4}{dx^3} = 0$$

and when $x = l_2 = 18$

$$A\,u_1 + B\,u_2 + C\,u_3 + D\,u_4 = v = e\,R\,\theta$$

$$A\,\frac{du_1}{dx} + B\,\frac{du_2}{dx} + C\,\frac{du_3}{dx} + D\,\frac{du_4}{dx} = 0$$

or, calling $A \div e\,R\,\theta = A''$, etc., these become
when $x = l_1 = 9$

$$\left.\begin{aligned} A''\,\frac{d^2 u_1}{dx_2} + B''\,\frac{d^2 u_2}{dx^2} + C''\,\frac{d^2 u_3}{dx^2} + D''\,\frac{d^2 u_4}{dx^2} = 0 \\ A''\,\frac{d^3 u_1}{dx^3} + B''\,\frac{d^3 u_2}{dx^3} + C''\,\frac{d^3 u_3}{dx^3} + D''\,\frac{d^3 u_4}{dx^3} = 0 \end{aligned}\right\}$$

when $x = l_2 = 18$ $\ldots\ldots (60)$

$$\left.\begin{aligned} A''\,u_1 + B'' + u_2 + C'\,u_3 + D''\,u_4 = 1 \\ A''\,\frac{du_1}{dx} + B''\,\frac{du_2}{dx} = C''\,\frac{du_3}{dx} + D''\,\frac{du_4}{dx} = 0 \end{aligned}\right\}$$

The coefficients on the left-hand side of Equation (60) are the same as for the case of the dam full, so that the numerical work for the solution is confined to the necessary alteration of the quantities on the right-hand side. Solving,

$$A'' = -23.575\,513$$
$$B'' = -\ 5.099\,821$$
$$C'' = -\ 7.560\,851$$
$$D'' = +19.048\,045$$

Observing that $A = A'' \times e\ R\ \theta$, u is determined, and therefore T and M, throughout for any specified temperature range θ. These must be superposed on the former results.

For the bending moments

$$M = C_1 \frac{d^2 u}{d z^2}$$

but since

$$z = a\ x$$
$$C_1 = \frac{1}{12} E_1 t^3$$
$$t = c\ z$$
$$a^2 = \frac{1}{12} \cdot \frac{E_1}{E_0} \cdot c^2 R^2$$

Therefore,

$$M = \frac{E_0 c^3}{c^2 R^2} z^3 \frac{d^2 u}{d x^2} = -\frac{E_0 c a^3}{R^2} x^3 \frac{d^3 u}{d x^2}$$
$$= \frac{E_0 9 e c a^3}{R} x^3 \frac{d^2 u''}{d x^2}$$

where

$$u'' = A''\ u_1 + B''\ u_2 + C''\ u_3 + D''\ u_4$$

In order to find the actual displacements, it is to be observed that the rise of temperature θ was supposed to have caused the dam to expand freely and uniformly throughout, so that the displacement at all points was $v = -e\ R\ \theta$. By imagining the dam forcibly brought back so that its base occupies the original position, by making $u'' = 1$ at all points of the base, the corresponding displacements u'' at all points of the dam were found. It is these which must be used for finding the stresses, but to find the displacements, deduct 1 from u'' at all points.

TABLE 5.—(*Continued.*)

$$\frac{d^2 u''}{d x^2} = A'' \frac{d^2 u_1}{d x^2} + B'' \frac{d^2 u_2}{d x^2} + C'' \frac{d^2 u_3}{d x^2} + D'' \frac{d^2 u_4}{d x^2}$$

VALUES OF x, IN FEET.

	9	10	11	12	13	14	15	16	17	18
$A'' \frac{d^2 u_1}{d x^2}$	− 2.796 049	− 4.033 623	− 5.259 896	− 6.437 739	− 7.528 934	− 8.494 770	− 9.296 672	− 9.896 856	− 10.258 975	− 10.348 799
$B'' \frac{d^2 u_2}{d x^2}$	+ 2.225 510	+ 2.094 110	+ 1.861 083	+ 1.524 835	+ 1.085 958	+ 0.547 308	− 0.085 941	− 0.806 270	− 1.603 822	− 2.466 436
$C'' \frac{d^2 u_3}{d x^2}$	+ 1.835 921	+ 1.345 446	+ 0.740 680	+ 0.029 355	− 0.777 723	− 1.666 860	− 2.621 685	− 3.623 086	− 4.649 892	− 5.678 620
$D'' \frac{d^2 u_4}{d x^2}$	− 1.265 382	+ 0.593 528	+ 2.655 747	+ 4.877 968	+ 7.210 718	+ 9.598 940	+ 11.932 686	+ 14.297 905	+ 16.477 297	+ 18.451 257
	+ 4.061 431	+ 4.033 074	+ 5.257 510	+ 6.432 158	+ 8.296 676	+ 10.146 248	+ 11.982 686	+ 14.297 905	+ 16.477 297	+ 18.451 257
	− 4.061 431	− 4.033 623	− 5.259 896	− 6.437 739	− 8.306 657	− 10.161 630	− 12.004 248	− 15.326 212	− 16.512 689	− 18.493 855
$\frac{d^2 u''}{d x^2}$	+ 0.000 000	− 0.000 549	− 0.002 386	− 0.005 581	− 0.009 981	− 0.015 382	− 0.021 562	− 0.028 307	− 0.035 392	− 0.042 598

TABLE 5.—Temperature Effect, 33-Foot Dam, Base Encastrée.

$$u'' = A'' u_1 + B'' u_2 + C'' u_3 + D'' u_4$$

Where

$$A'' = A \div e\ R\ 9$$

$$u'' = u \div e\ R\ 9$$

$$A'' = -23.575\ 513$$

$$B'' = -\ 5.099\ 821$$

$$C'' = -\ 7.560\ 851$$

$$D'' = +\ 19.048\ 045$$

The deduction of 1 is made in order to represent the fixed base, leaving the rest of the cylinder expanded by the rise of temperature.

Values of x, in Feet.

	9	10	11	12	13	14	15	16	17	18
$A'' u_1$	+ 85.235 093	+ 97.364 942	+ 105.462 106	+ 108.303 405	+ 104.714 182	+ 93.606 470	+ 74.017 659	+ 45.148 984	+ 6.403 805	− 42.578 652
$B'' u_2$	− 0.996 437	+ 8.521 724	+ 20.125 517	+ 33.581 782	+ 48.554 322	+ 64.604 493	+ 81.194 080	+ 97.690 462	+ 113.374 127	+ 127.448 588
$C'' u_3$	+ 21.927 503	+ 36.302 750	+ 52.013 907	+ 68.456 847	+ 84.921 154	+ 100.600 885	+ 114.608 285	+ 125.990 141	+ 133.746 796	+ 136.858 391
$D'' u_4$	− 106.098 437	− 141.981 910	− 177.254 882	− 209.858 750	− 237.575 408	− 258.076 723	− 268.979 443	− 267.905 191	− 252.544 314	− 220.723 288
	+ 107.162 596	+ 142.189 416	+ 177.601 530	+ 210.342 034	+ 238.189 658	+ 258.811 848	+ 269.820 024	+ 268.829 587	+ 253.524 228	+ 264.301 929
	− 107.094 874	− 141.981 910	− 177.254 882	− 209.858 750	− 237.575 408	− 258.076 723	− 268.979 443	− 267.905 191	− 252.544 314	− 264.301 930
u''	+ 0.067 722	+ 0.207 506	+ 0.346 648	+ 0.483 284	+ 0.614 250	+ 0.735 125	+ 0.840 581	+ 0.924 396	+ 0.979 914	+ 0.999 999
$(u'' - 1)$	− 0.932 278	− 0.792 494	− 0.653 352	− 0.516 716	− 0.385 750	− 0.264 875	− 0.159 419	− 0.075 604	− 0.020 086	− 0.000 001

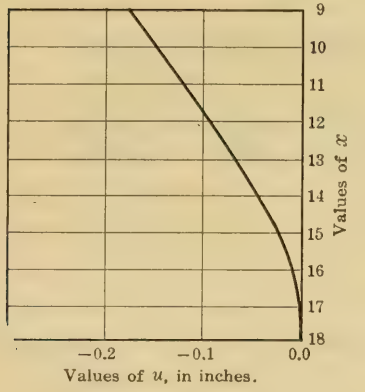


FIG. 10.—DISPLACEMENT CURVE, u

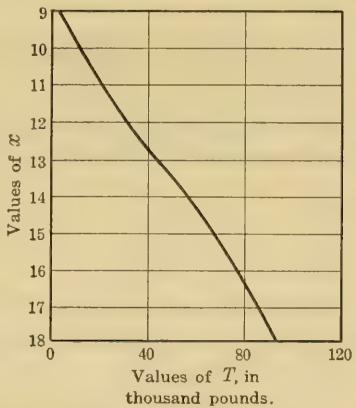


FIG. 11.—ARCH THRUST T

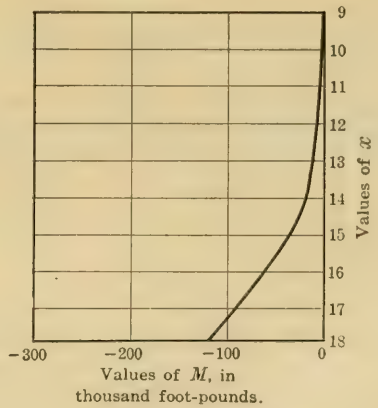


FIG. 12.—BENDING MOMENT M

FIGS. 10, 11, 12.—DISPLACEMENT, THRUST AND BENDING MOMENT DUE TO TEMPERATURE.

To find the thrust

$$T = \frac{E_0 t}{R} u = \frac{E_0 c a}{R} x u = E_0 \theta e c a \times x u$$

For example, if $\theta = 20°$ Fahr., $e = 0.000\ 005\ 5$ $R = 135$, then $e R \theta = 0.015$ ft., say, $E_0 \theta e c a = \frac{2}{3} (288 \times 10^6) \times 20 \times .000\ 005\ 5 \times \frac{1}{15} \times \frac{11}{3} = 5160$, say

$$\frac{E_0 \theta e c a^3}{R} = 5160 \times \frac{a^2}{R} = 5160 \times \left(\frac{11}{3}\right)^2 \times \frac{1}{135} = 514, \text{ say.}$$

TABLE 6.—Temperature Effect, Base Encastrée, Crest Free, Dam Empty.

x	z, in feet.	d, in feet.	u, in inches.	T, in pounds per foot height.	M, in foot-pounds per foot width.
9	33.0	0	— 0.168	3 150	0
10	36.6	3.6	— 0.143	10 700	— 300
11	40.3	7.3	— 0.118	19 700	— 1 600
12	44.0	11.0	— 0.093	29 900	— 5 000
13	47.6	14.6	— 0.069	41 200	— 11 300
14	51.3	18.3	— 0.048	53 100	— 21 700
15	55.0	22.0	— 0.029	65 100	— 37 400
16	58.6	25.6	— 0.013	76 300	— 59 600
17	62.3	29.3	— 0.004	86 000	— 89 400
18	66.0	33.0	— 0.00	92 900	—127 700

APPENDIX V.

DEFLECTIONS IN THE WOOLING DAM.

The deflection observations for the Wooling Dam were somewhat rough, as there was considerable oscillation in the plumb line on account of wind. In making the observations, a plumb line was hung from a small triangular notch in an angle iron built into the concrete at crest level, and the distances from the plumb line to the wall were measured at each 5-ft. interval on June 10th, 1916, when the dam was empty, and again on March 28th, 1917, when the dam was nearly full. A comparison of calculated with observed deflections is given in Table 7.

TABLE 7.—Comparison of Calculated and Observed Deflections in the Wooling Dam.

Depth below crest.	Displacement, in inches, due to water pressure—dam full —, + when down stream.	Displacement, in inches, due to rise of temperature of 20° Fahr.—dam empty.	Resultant — two preceding columns.	Depth below crest, in feet.	Observed displacement, in inches, water 1 ft. 8 in. below crest March 28th, 1917.
0	+ 0.19 in.	— 0.17 in.	+ 0.02 in.	0	0
3 ft. 8 in.	+ 0.17	— 0.14	+ 0.03		
7 ft. 4 in.	+ 0.14	— 0.11	+ 0.03	5	+ 1/16
11 ft. 0 in.	+ 0.12	— 0.09	+ 0.03	10	+ 1/8
14 ft. 8 in.	+ 0.09	— 0.07	+ 0.02	15	+ 1/8
18 ft. 4 in.	+ 0.07	— 0.05	+ 0.02		
22 ft. 0 in.	+ 0.04	— 0.03	+ 0.02	20	+ 1/8
25 ft. 8 in.	+ 0.02	— 0.01	+ 0.01	23*	0
29 ft. 4 in.	+ 0.01	— 0.00	+ 0.01		
33 ft. 0 in.	+ 0.00	— 0.00	+ 0.00		

* Ground level is 23 ft. below crest.

The temperature was not recorded during construction but most of the work was done during the months of March to June, 1916. Table 8 gives the official records of the mean monthly temperature at Macedon, about 3 miles from the dam and at about the same height above sea level. The temperature of the water was observed on November 2d, 1916, to be 13° Cent. = 55° Fahr., and the estimated temperature on March 28th, 1917, was 60° Fahr. The water was then cool, but not cold.

TABLE 8.—Mean Temperature, in Degrees Fahrenheit, at Macedon, Victoria, Australia.

January	65.1	May	48.7	September	48.8
February	64.7	June	45.1	October	53.6
March	60.8	July	43.2	November	58.7
April	54.7	August	45.4	December	63.0

APPENDIX VI.

ACKNOWLEDGMENTS OF ASSISTANCE.

It is desired to acknowledge the assistance given in the investigation by Mr. J. H. Michell (Melbourne) who reduced Equation (30) to the general form, Equation (34), and gave the four series solutions of this equation which have been tabulated as Michell's functions. Acknowledgment must also be made of the assistance given by the writer's assistants Mr. A. French* and Mr. H. M. Sherrard who carried out the laborious work of tabulating Michell's functions and carefully checked every result; also of the courtesy of Dr. Baldwin, the Acting Government Astronomer of Victoria (Australia), for his assistance in suggesting the best method of calculating the tables and for allowing the use of his eight-figure calculating machine. Finally, mention must be made of the valuable suggestions as to arrangement of subject-matter and as to points requiring further elucidation made by the Publication Committee of the Society.

* Died in active service.

DISCUSSION

Mr. Wernecke.

CHAUNCY WERNECKE,* ASSOC. M. AM. SOC. C. E. (by letter).—The author deserves much credit for working out the needed modification and development of the cylinder theory which has been applied to arched dams. For practical applications, the simple formula for the thickness of an arch dam, $t = \frac{R\ p}{q}$, in which q is the allowable unit stress, does not need revision at the present time, as it has not yet been generally accepted as giving conservative results in dam design. It needs a fuller analysis of its application to dams, which, of course, should be checked by tests and deflection measurements. The author's labors are a worthy addition to this analysis.

That the cylinder theory does give conservative results is shown by Fig. 13 which points out the effect of the radius on the thickness of an arch dam.

It will be seen that the graph of this formula lies on the safe side. A curve giving the thickness of a curved dam having the same factor of safety as the most conservative gravity type would be of the character suggested by the dash line in the diagram. This dash line is merely a suggestion of an approximation to what extended theory and tests may eventually develop. Such a curve, of course, could not be expressed by a single equation, because a change of stress action would occur according to conditions and as the thickness passed certain limits. Where gravity action ceases and arch action begins in the stress action of dams curved in plan, and where arch action ceases and plug or "bracket" action begins in short dams, are problems yet to be solved.

The Arrowrock Dam is curved in plan to a radius of 662 ft., but is of the gravity type. An arch dam with such a radius for a unit stress not exceeding 40 000 lb. per sq. ft. would have a thickness of 103 ft. at a point 100 ft. below the crest. What engineer would not discard the cylinder theory as too conservative should he apply it to the Arrowrock Dam, but, on the other hand, how many engineers would accept its results for the same dam if it was possible to make the radius 300 ft. instead of 662 ft.? Is it not time that all engineers come to realize that it is visually impossible to compare, for relative factors of safety, the profiles or cross-sections of arch dams with those of the gravity type, or with arched dams themselves? The radius is an invisible factor in the cross-section and as important as the thickness itself.

The need for further deflection measurements on existing dams and tests on models is now quite apparent. Unfortunately, the cost

* Seattle, Wash.

of such tests appears to be very great, but the writer believes this is more apparent than real, when considered in the light of the importance of the problem. Mr. Wernecke.

The fact that there are only a few dams compared to the number of such structures as buildings, bridges, or railroad tracks, may account for the slowness with which this subject has been taken up; but the reason of mere lack in numbers is a primitive one, and is offset by the fact that the destruction due to the failure of a dam may be very great. There have been no failures of arch dams to date, but, nevertheless, the factor of safety in these dams must be determined, then their economies will be evident to all.

Adequate tests may not easily be handled by any one institution. In fact, it is a task of co-operation, subsidized by the Government.

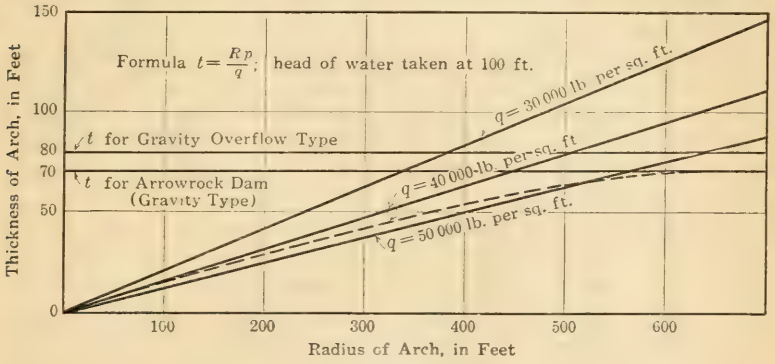


FIG. 13.

Such a testing laboratory would require only a modest sum, as the writer wishes to point out. The cost would depend chiefly on its location and size.

Assuming an easily available site and average size of structure, the cost of a testing plant like that shown in Fig. 14, would be about $250 000. This would include a suitable pump, a portable truck crane, the requisite recording instruments, two traveling bridges, a building, and miscellaneous installations. The operation of the laboratory would be effected as a matter of course. No doubt many of the dams tested would have to be removed from the canyon by blasting.

As a progressive people, Americans have built expensive observatories and telescopes which have assured us that potatoes will grow if planted in the dark of the moon. Why not now build a dam-testing canyon so that engineers will know that better and cheaper dams can be built to aid those very same potatoes in growth and in cooking? In the United States, California has taken the lead in the development

Mr. Wernecke.

of dams and is the home of the arch dam. The climate there is ideal for an outdoor laboratory, economic sites for such a laboratory are numerous, and, furthermore, capable engineers would be found at hand.

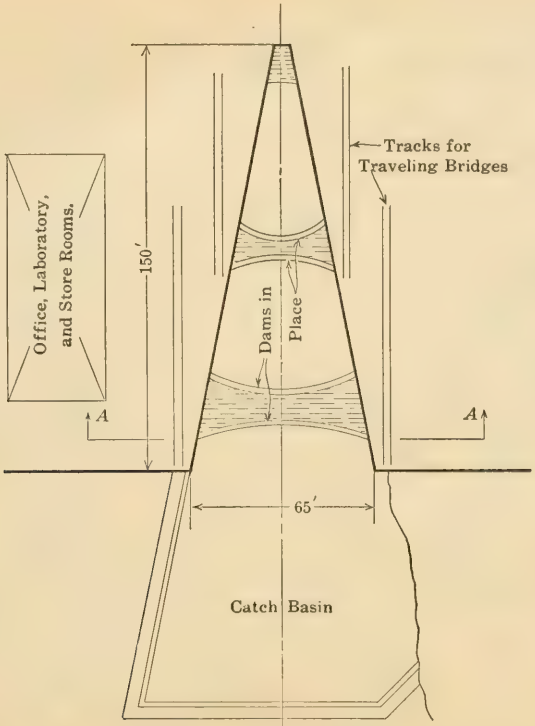
That the cost of $250 000 for such a laboratory could be saved in a single dam is easily possible, and this amount is a small part of the outlay on a dam costing from $4 000 000 to $5 000 000. The writer has projected designs of dams for heights from 200 to 450 ft., for which the sites assumed were, in most cases, suitable for an arch or could be built for an arch by using gravity tangent wings. For some of these sites there is a difference in cost of from $500 000 to $1 000 000 in favor of an arch dam as compared to a gravity structure. The final proof that arched dams are safer and better than dams of the gravity type will be of great economic value in the immediate future. The recurring coal strikes, in fact, all labor troubles, development of industries, practicability of electrification of trans-continental railroads, reclamation, and irrigation, are calling for water development more forcefully than ever before, and the ultimate development of every water resource comes in as a guiding limit. This brings up the new problems involved in very high dams—dams which may be even as great as 1 000 ft. in height.

The forerunners of these high dams are already noted. One in India is projected to be 395 ft. high. The Colorado River Dam will be 400 ft. or more in height, and the American Falls Dam, in Idaho, will be a very high one. In view of the fact that comparatively few men are engaged on the construction of dams at any one time, it would seem that no institution other than a Society like this could afford to advance the knowledge on these unprecedented structures, and, accordingly, the Society should proceed actively in this matter. Possibly the best way to present these problems would be apart from any actual structure, which was the method used by the late C. L. Harrison, M. Am. Soc. C. E., in presenting the question of "Provision for Uplift and Ice Pressure in Designing Masonry Dams".*

The use of arch dams is subject to limitations, but at the present time most of these are due to personal opinions. Considering the question of limit of height, Fig. 15 is of interest. This diagram shows the limits in height that result if one holds to the opinions of two early French engineers. M. Delocre (1866), holds that a curved dam would act as an arch if its thickness at the base does not exceed one-third its radius, and M. Pelletreau (1876), believes that this thickness should not exceed one-half its radius.

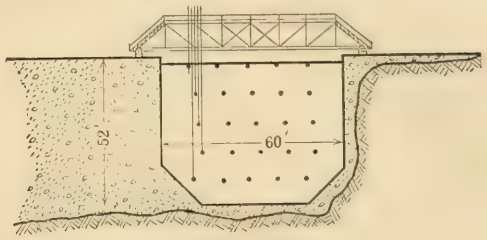
In a dam, 400 ft. or more in height, the problem as to when arch action ceases is of serious moment. That unprecedentedly high dams

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 142.



PLAN OF DAM-TESTING CANYON

Showing 5 Dams in Place Ready for Tests.



SECTION A-A

Showing bridge with set of levers for deflection measurements. The Dam here is 60 ft. long by 50 ft. high. Radius = 100 ft.

FIG. 14.

Mr. Wernecke.

in the immediate future are a certainty, does not need to be argued in this discussion, but engineers with influence in the larger institutions, especially, and others as well, should give substantial encouragement to such analyses as those presented by the author and should see that extended measurements and tests are carried out. It may not be out of place either to call attention to the fact that the water pressure alone at the heel of a 400-ft. dam is 25 000 lb. per sq. ft. and 37 500 lb. per sq. ft. for a 600-ft. structure. This means that the

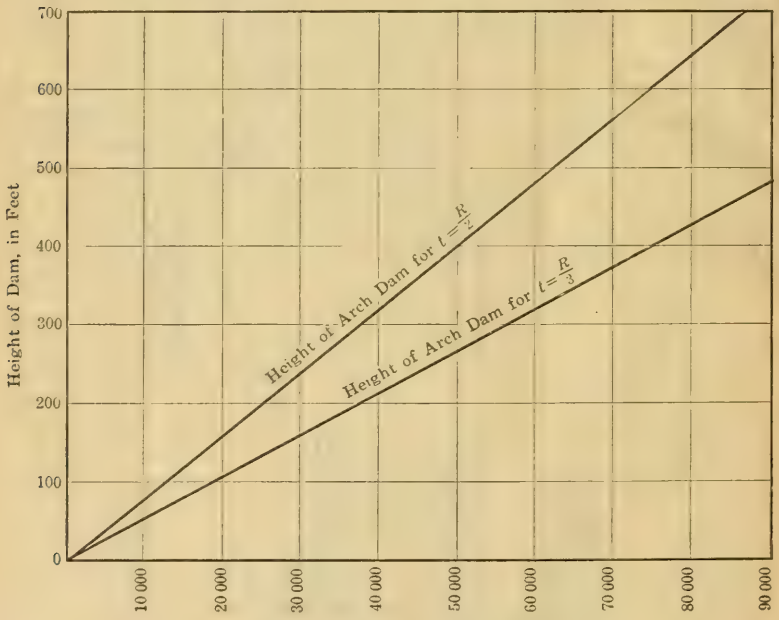


FIG. 15.

present allowed maximum unit stresses must be higher in the future, which means further that tests of unprecedented character must be made.

The author's interpretation of the deflections of the Barren Jack Creek Dam are helpful at this time. Heretofore, too many engineers have referred to these deflection measurements as proof that intricate investigations are of little use. If a concrete arch dam is cast in the coldest weather, subsequent temperatures stresses will cause it to deflect up stream. Does this not warrant writing into construction specifications the requirement of pouring a small vertical central portion, which may be called a key, during the coldest weather

practicable? Between the lines, this seems to have been suggested by the author. Mr. Wernecke.

In conclusion, the writer will state that he has gone into the many intricate, theoretical investigations of determining stresses in arch dams and even projected some himself, but he feels that this phase of the problem has been extended quite beyond practical use; that the burden of support is now on those who can furnish extended deflection measurements, and most of all on the Society, the Government, and on leading technical institutions for adequate tests.

L. R. Jorgensen,* M. Am. Soc. C. E. (by letter).—The author goes into considerable mathematical detail in his calculations of arch dams. Mr. Jorgensen.

The correct calculation of the various stresses in such dams is only possible when correct assumptions can be made. To make fairly complete calculations of an arch dam is always difficult, but to make correct assumptions is still more difficult. It is not much use to complicate the calculations and expect more accurate results when the assumptions are not correct in the first place.

It is the writer's opinion, based on his experience, that in order to make any assumption hold good for a curved wall in a canyon acting like an arch, it is necessary to fill the contraction joints with grout or concrete during cold weather, with reservoir empty. If that is successfully done, arch action can be expected to be nearly perfect during warm weather, with reservoir full, as hardly any deflection of the crown in a down-stream direction will then take place.

If no deflection takes place, that is, if the decrease in length of the arch due to load should just be compensated for by the increase in length due to the higher temperature (reservoir full, during the warm season), the simple arch formula would hold true, including its modifications in regard to stresses along the up and down-stream faces.

The simple arch formula does not hold true when the arch is thick and flat, and the abutments are closer together; but in such case neither does any other arch formula. The structure becomes a (straight) beam, and the beam action carries the load because much less deformation is required that way.

In such a case, there is only one way to make the curved beam act mainly as an arch, that is, by forcing grout under pressure into the contraction joints. At the bottom, this is difficult or impossible because the contraction joints do not open up close to the bottom on account of the shearing strength of the concrete at the foundation and on account of the action of Poisson's ratio.

The lateral deformation (Poisson's ratio) of the concrete at and near the foundation in high dams is sufficient to give the dam a push in an up-stream direction, so to speak, like a rise in temperature, and

* San Francisco, Cal.

Mr. Jorgensen.

any theory that neglects this factor cannot be considered to be accurate. The lateral deformation of the concrete at the bottom, caused by the weight of material above, should be sufficient to prevent the dam from "lifting off its seat on the up-stream side", due to this tendency of pushing it in an up-stream direction initially.

The beam action will have the same effect, that is, prevent the dam from lifting off its seat, by minimizing the actual deformation in a down-stream direction, with reservoir full.

Any dam calculation which is intended to be complete, must also make provision in some way for the action of the time factor in connection with Young's modulus of elasticity. As long as the stresses in the concrete are kept low, the time factor has no effect, but as soon as the stresses increase toward the breaking point, the effect is such as to change Young's modulus of elasticity, provided, however, that it has taken the stresses at least 14 days to develop fully. This, perhaps, will always be the case for dam structures, as it generally takes much longer than 14 days to fill the reservoir and create the maximum stresses.

It has been fully borne out in practice and by numerous tests, that the deformation of concrete, if highly stressed, is, say, three times greater after a time interval of 14 days or more, than it was just after the load had been applied (Young's modulus). Therefore, it will be found that, in spite of the fact that ordinary formulas may show that a dam structure is or should be cracked or lifted off its seat and it has not done so, the material has stretched more.

What is shown by practice must be taken as being correct, and the ordinary formulas, therefore, are wrong, when it comes to extreme conditions. The concrete will compress or stretch itself much more than in accordance with Young's modulus whenever a time interval is imposed between the occurrence of minimum and maximum stresses, and this is always the case in dam structures.

Mr. Cain.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author has made a very valuable contribution to the mathematics of the subject of arched dams. After referring to the ordinary cylinder theory, and noting its inadequacy, he modestly states that:

"This is an attempt to carry the approximation of the cylinder theory a step further by taking account of the shear across a horizontal plane and the corresponding bending moment about a horizontal tangent to the cylinder."

The theory, in its results, is thus closely in line with that to be examined in this discussion, where the effort is made to co-ordinate—at least, approximately—the deformations of the dam acting as a vertical cantilever with its deformations where it acts as a horizontal arch.

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Mr. Cain.

It is recognized that neither theory is complete, since there is not only shear in a radial direction between the elementary horizontal arches into which the dam is supposed to be divided, but likewise between the arches in an axial direction (perpendicular to the radius), since the shortening of each horizontal arch, when under stress, is different from that of the arch below it, so that each exerts a shearing stress on the next one under it. The second set of axial shearing forces tends to make the dam more rigid than in the case where the horizontal arches are supposed to be free to slide in an axial direction on each other, so that they contribute essentially to the rigidity of a dam, particularly in a V-shaped valley with rigid abutments, hence neglecting them is on the side of safety. It is fortunate that this is so, for to include this second set of shearing forces in an analysis would doubtless make it too complex for solution. In fact, the author's analysis, omitting such forces, is sufficiently complicated as it stands.

The author's analysis, subject to the "reservations" stated in his paper, is complete; in fact, it is more so than any analysis hitherto made, and gives not only the deflections at various depths for points on the dam farthest up stream, but likewise the arch thrusts and moments for temperature changes, as well as for water pressure.

In Appendix II, a short approximate formula is deduced for the radial deflection at the crown of a segmental arch, fixed in position, but free to turn at the ends, on the hypothesis that the arch can slide freely on its base. In reality, each horizontal arch of the dam, to which this formula is applied, is not free at the ends; neither is it "fixed", since the arch is, perhaps, never secured to the sides of the valley in such fashion that the ends can be regarded as encastrée. Further, each arch is not free to slide over the one below it; hence, from both causes, the crown deflection, as given by the formula, is somewhat too large; but, from lack of knowledge as to the true conditions, it may be provisionally accepted, particularly because of its simple form. If thought advisable in the future, the coefficient $\frac{3}{2}$ of this Equation (51) can be changed, to conform to further knowledge, throughout the main mathematical development of Appendix I, which is elegant and rigorous, and leads to the final formulas to be used in practice.

When the dam is reinforced horizontally so that a vertical section of concrete of unit area contains the area, $p_0 \times 1$, of steel, the author properly writes the modulus of elasticity in a horizontal direction, for a homogeneous material, $E_0 = E\ (1 + 15\, p_0)$; for, then, the total stress on the cross-section for the homogeneous material, for a unit

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shortening, f, is $E_0 f$, and for the actual section, the stress is $E f$ on the concrete, and $E f \times 15 p_0$ for the steel, or a total of $E f (1 + 15 p_0)$. On equating these two expressions and dividing by f, we are conducted to the formula for E_0 as given. Similarly, the formula for E_1 is derived. The use of these formulas evidently conduces to simplification.

In Appendix I, Michell's functions, with the corresponding numerical values of Table 1, are introduced for the first time. The solution of the arched dams of wedge-shaped cross-section is effected by aid of such functions, as illustrated in the numerical applications to the Wooling Dam, given in Appendices III and IV. The author carefully cautions the reader, in using the tables, to retain six or seven significant figures, on account of the very small resultants found when the functions are properly combined. The writer found the same drawback in computing unit stresses in any direction in the interior of a straight masonry dam,* and was forced to use seven-place logarithmic tables to attain correct results. From what the author says, such a table would have to be used in solving the equations given in Tables 2, 3, and 5, unless a 6 or 8-figure calculating machine is available.

The cylinder formula, $T = p R$, which is utilized in this investigation, is not exact. The strict solution is exactly that derived for thick cylinders, as given (say) in Merriman's "Mechanics of Materials", 10th edition, Articles 149 to 153. Thus, for any horizontal arch of a vertical cylinder, subjected to a water pressure, p per square foot, acting radially and inwards, let,

$r =$ outer radius of cylinder;
$r_1 =$ inner radius of cylinder;
$x =$ distance from center of cylinder to a point in the arch ring;
$S =$ circumferential unit stress, or stress on a radial section which acts perpendicular to the radius.

Then, using Lamé's formula as sufficiently near for the purposes (the other formulas involve Poisson's ratio), we have, since the pressure on the inner surface of the cylinder is zero,

$$S = p \, r^2 \frac{1 + \frac{r_1^2}{x^2}}{r^2 - r_1^2}$$

Thus, S does not vary linearly from $x = r_1$ to $x = r$.

If $t = r - r_1$, the thickness of the arch ring is small compared to r_1, then the unit circumferential thrust at $x = r$ and $x = r_1$ respectively, are, very nearly,

$$S_0 = \frac{p \, r}{t} \quad \text{and} \quad S_1 = \frac{r}{r_1} S_0$$

* *Transactions*, Am. Soc. C. E., Vol. LXIV (1909).

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The value of S_0 or the thrust per unit at the outer surface, is the well-known value for a linear arch. Continuing to regard t as very small as compared with r_1, S can be supposed to vary linearly, so that the arch thrust, T, acting on an area, $t \times 1$, is very nearly

$$T = \frac{1}{2} t (S_0 + S_1) = S_0 t \left(\frac{r_c}{r_1}\right) = p r \left(\frac{r_c}{r_1}\right)$$

Where $r_c = \frac{1}{2}(r + r_1)$ is the radius of the center line or axis of the arch.

The usual cylinder formula, $T = p r$, is, thus, not exact, but sufficiently near in most cases. The error is only 1% for the Wooling Dam (Appendix III). The closer approximation might be called for in the case of a thick concrete tank.

The author, in Appendices III to VI, gives the complete numerical solution for the Wooling Dam, which is 33 ft. high, the crest width being 2.2 ft., the bottom width 4.4 ft., and the radius of the up-stream face being 135 ft. The water surface is supposed to be level with the crest, and the dam is supposed to be fixed at the base. The angle of the sector is about 120° at the crest level. In Table 4, for water pressure only, and in Table 6, for a rise of 20° Fahr. in temperature, for reservoir empty, there are given, as the final results of computation for various depths, d, below the crest, the deflection, u (in inches) of the center, the thrust, T, on an arch 1 ft. in height, and the moment, M, per foot width.

Now, consider a cantilever of the dam, enclosed between two radial vertical planes, which make a very small angle with each other. As this angle approaches zero indefinitely, without ever becoming zero, the planes approach parallelism indefinitely and the water pressures on the cantilever all approach parallelism. By the theory of limits, all these pressures can be regarded exactly as if they acted perpendicular to a plane tangent to the upper face of the cantilever along its central vertical generator, which will be taken through a point on the dam farthest up stream. The unit water pressures on this cantilever are precisely the same as for a cantilever contained between two parallel vertical planes, 1 ft. apart, and perpendicular to the tangent plane mentioned to be taken as the water face of the cantilever. For such a cantilever, the unit water pressures become total pressures per foot height and, in what follows, such a cantilever, 1 ft. wide, will be considered.

The dam is supposed to be divided into a series of horizontal arches, each 1 ft. in depth, the mid-plane of each being at a depth, d, as given in Table 4.

In Table 9 are given, for successive depths in feet below the crest, d, the thrust, T, taken from the author's Table 4, the unit water

Mr. Cain. pressure, p, acting on the arch, found by aid of the formula, $T = p\,R$, the unit water pressure, $p' = 62.5\,d$, acting on the dam, and, finally, the pressure on the cantilever, $p' - p$.

In using the formula, $p = \frac{T}{135}$, it is tacitly assumed that each horizontal arch is free to move; in other words, the radial shear acting between adjacent arches is ignored. The author's analysis includes this shear, so that the numerical values for p and $p' - p$, given in Table 9, are only approximate. The interpretation of these figures is as follows:

From $d = 7.3$ to $d = 33.0$, the horizontal load on the cantilever, $p' - p$, is positive and acts down stream, so that both the arches and cantilever are acted on by forces directed down stream at these levels. At $d = 0$, or $d = 3.6$, $p' - p$ is negative, so that the load on the cantilever acts up stream. Thus, at the crest, the cantilever acting alone under the water pressure would deflect more than a horizontal arch of the dam there. This excess deflection is resisted by the arch, thus supplying a force acting up stream on the cantilever. Similarly, at $d = 3.6$, the arch reacts on the cantilever with a force, acting up stream, of 132 lb. per sq. ft. At $d = 5.4$ ft., there is no pressure on the cantilever, the water pressure at that point being carried entirely by the arch.

TABLE 9.

d, in feet.	T, in pounds per foot of height.	$p = \frac{T}{135}$, in pounds per square foot.	$p' = 62.5\,d$, water pressure, in pounds per square foot.	$p' - p$, pressure on cantilever, in pounds per square foot.
0	49 000	363	0	− 363
3.6	48 200	357	225	− 132
7.3	45 800	340	456	+ 116
11.0	41 100	305	687	+ 382
14.6	35 500	263	912	+ 649
18.3	27 600	205	1 144	+ 939
22.0	18 700	139	1 375	+ 1 236
25.6	9 900	73	1 600	+ 1 527
29.3	2 900	21	1 831	+ 1 810
33.0	0	0	2 062	+ 2 062

When the pressure, $p' - p$, on the cantilever is minus, it acts up stream; when $p' - p$ is plus, it acts down stream.

Of course, for the rise of temperature of 20° Fahr., with the reservoir empty, the deflection of an arch is up stream, so that its pressure on the cantilever is everywhere in an up-stream direction. The investigation for the combined water pressure (reservoir full) and a temperature rise of 20° Fahr. (see Tables 4 and 6) is made exactly as the above, and shows similar results, except that the point of no pressure on the cantilever is about 8 ft. below the crest, at which point the arch alone carries all the water pressure.

Recurring now to the case of the reservoir full, but no temperature change, from the unit stresses, $p' - p$, given in Table 9, compute the total loads on successive segments of the cantilever, as shown in Fig. 16.* For greater accuracy in finding the moments, m, about the points, $d = 3.6$, $d = 7.3$, etc., given below, the loads were supposed to be applied at the centers of gravity of the trapezoids or triangles, measuring quantities only to the tenth of a foot. Mr. Cain.

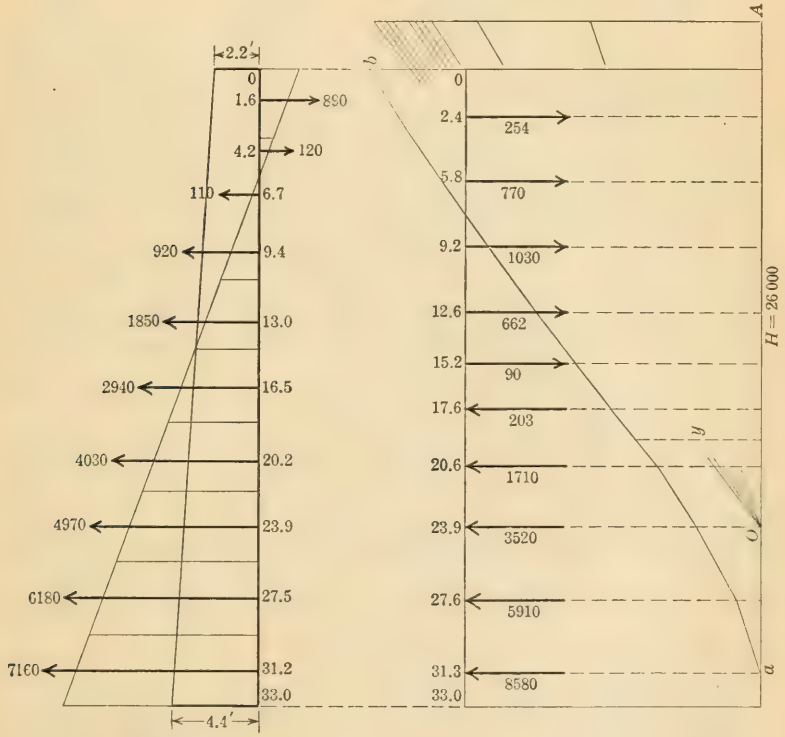

FIG. 16. FIG. 17.

On taking moments of these loads about points at the depths d ft. below the crest, the moments m on the cantilever at the successive depths are derived. The author's moments, M, taken from Table 4, are placed alongside for comparison.

The difference between m and M is given as a percentage of M in the last column of Table 10. It varies from -5.1% to $+8.1\%$, being very small for the three lowest points.

* The line limiting the unit stresses is very nearly straight.

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TABLE 10.

d	m	M	Percentage of algebraic increase.
0.0	0	0	.....
3.6	— 1 780	— 1 900	+ 6.3
7.3	— 5 379	— 5 500	+ 2.2
11.0	— 7 237	— 7 500	+ 3.5
14.6	— 4 205	— 4 000	— 5.1
18.3	+ 8 006	+ 7 400	+ 8.1
22.0	+ 33 057	+ 32 600	+ 1.4
25.6	+ 73 330	+ 73 200	+ 0.2
29.3	+ 135 551	+ 134 800	+ 0.6
33.0	+ 222 401	+ 220 900	+ 0.7

The percentage differences being quite small, the inference may be drawn that, ignoring the radial shear between the arches, the moments on the cantilever are only slightly affected. This proving encouraging, the attempt will next be made to compute deflections and compare with the values of u given by the author for the Wooling Dam. The Mohr graphical method will be used*. By this method, the values of $\frac{m}{I}$ should be plotted as horizontal ordinates at the corresponding points of the vertical side of the cantilever. Here, $I =$ moment of inertia of the beam where the thickness is t, so that $I = \frac{1}{12} t^3$. On account of the labor involved in a strict construction, and considering that t does not vary greatly in the vertical segments, Δx, taken not more than 3.7 ft., an average value of t was used for each Δx, so that the moment curve alone was drawn, its area for each base, Δx, computed (designated by $M \Delta x$, where M is an average moment), and the center of gravity of that area found.† Then, omitting for the present the factor, $\frac{1}{12}$, the quantities $\frac{M \Delta x}{t^3}$ were computed and laid off as horizontal forces acting on the cantilever at the corresponding centers of gravity mentioned. The negative forces will act to the right, the positive forces to the left. In Fig. 17, these forces are drawn in position and their magnitudes given. The author gives the value of the modulus $E_1 = 2\,600\,000 \times 144$ lb. per sq. ft.; hence, all lengths being in feet, the deflection at any point, in feet, is given by the formula:

$$\frac{I}{E_1} \sum \frac{M \Delta x}{I} x = \frac{12}{E_1} \sum \frac{M \Delta x}{t^3} x;$$

* Also, see Morley's "Strength of Materials", Articles 82-83.

† Note here that M is the average of m (as computed by the writer and given in Table 10) on the height, Δx. It is not the author's M.

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therefore, the deflection in inches is:

$$\Delta_x = \frac{12 \times 12}{2\,600\,000 \times 144} \sum \frac{M\ \Delta\ x}{t^3}\ x = \frac{1}{2\,600\,000} \sum \frac{M\ \Delta\ x}{t^3}\ x.$$

In this formula, for any point on the cantilever, x represents the vertical distance from the point to any load, $\frac{M\ \Delta\ x}{t^3}$, below that point, and the summation of the products, $\frac{M\ \Delta\ x}{t^3}\ x$, must include all loads between the point and the base of the cantilever. This summation is easily effected graphically.

Thus, in Fig. 17, assume any pole, O, and lay off, to any convenient scale, vertically, $O\ A = 26\,000$ and from A, to the same scale, lay off horizontally to the left, the loads $\frac{M\ \Delta\ x}{t^3}$, the numerical values of which are 8 580, 5 910, etc., in order; the loads acting to the left being laid off to the left, those acting to the right being laid off to the right. The equilibrium polygon, $a\ b$, is then drawn in the usual manner, and by its known principle, $\sum \frac{M\ \Delta\ x}{t^3}\ x = H\ y$, where y is the ordinate to the equilibrium polygon at the point considered, measured to the scale of distance, in feet. Substituting this value in the preceding equation, we have finally for the deflection in inches at the point where the ordinate, y, is measured,

$$\Delta_x = \frac{y}{100}\ (\text{inches}),$$

where y is measured to the scale of distance, in feet. The values of the deflections thus derived, for the values of d given, are compared with the deflections, u, given by the author in Table 4.

TABLE 11.

d, in feet.	Δ_x, in inches.	u, in inches.	Percentage of excess.
0	0.206	0.188	9.6
3.6	0.182	0.166	9.6
7.3	0.157	0.143	9.8
11.0	0.130	0.118	10.2
14.6	0.102	0.093	9.7
18.3	0.076	0.068	11.8
22.0	0.046	0.042	9.5
25.6	0.023	0.021	10.0
29.3	0.006	0.006	0.0
33.0	0.000	0.000	0.0

The percentage difference is greater than that found in comparing moments. Part of this is due to errors already found in m and part

Mr. Cain. to assuming an average value of t for each segment, Δx, for reasons given previously. To get quite close results, the values of $\frac{M \Delta x}{t^3}$ should be laid off as ordinates, and then using twenty or thirty divisions instead of the ten here adopted, the loads can be computed, etc., and the construction proceeded with as before.

However, the results are again encouraging; for, even if the error amounted to 10%, the approximate method which ignores the shear between the arches is sufficiently exact for a practical solution, considering the many uncertainties in the assumptions concerning curved dams as regards the degree of "fixity" of the base and abutments, the shape of the valley, the proper value for the modulus of elasticity, and, above all, the temperature changes, which cause very high stresses. Then, it will be recalled that the axial shear between the horizontal arches has been ignored in all theories hitherto presented—this fortunately being on the side of safety—so that it seems fair to conclude that a solution that regards the horizontal arches as free to slide on each other should give practical results.

If a simple solution of this character (perhaps partly graphical) co-ordinating perfectly the deflections of arches and cantilevers could be devised, it would prove to be a boon to the practical engineer. For all approximate solutions, the author's paper will prove a valuable standard for comparison, and deserves to become a classic.

Mr. Smith. B. A. Smith,* M. Am. Soc. C. E. (by letter).—In his discussion, Mr. Wernecke has suggested the construction of a large-scale testing laboratory. Would it not be sufficient to construct small-scale models and apply Newton's Principle of Similitude?† It should be remembered that a test to destruction is of no scientific value, for all the elastic equations used are based on Hooke's law—strain varies as stress—and this law breaks down long before the structure fails.

Mr. Jorgensen points out that the correct calculation of the various stresses is only possible when correct assumptions can be made. He might safely have gone a step further and stated, which is quite true, that if complete account is taken of all the actual conditions, a mathematical solution of an engineering problem is impossible.

All that was attempted in the writer's paper was to obtain a solution for the stresses in an ideal case, which is never realized in practice, namely, a complete thin cylinder of uniform material obeying Hooke's law and subjected to water pressure. The approximate solution suggested for the segmental dam is purely tentative. How closely the ideal conditions of this problem are complied with in practice

* Melbourne, Victoria, Australia.

† See Newton's "Principia", Book 2, Section 7, Proposition 32; Routh's "Rigid Dynamics" (1882 Edition), p. 286; also, *Transactions*, Victorian Inst. of Surveyors, Vol. V (1900–09), p. 17.

must be left to the judgment of the engineer who is dealing with the case. Mr. Smith.

In reference to Professor Cain's discussion, the writer would state that it was tacitly assumed, though not expressly stated, that the cylinder is thin, that is, the ratio of the thickness to the radius is small.

The difficulty of determining stresses by graphical methods is very great, a careful scrutiny must be made of the equations on which a graphical method is based in order to insure that fundamental elastic conditions (such, for example, as St. Venant's strain relations) are satisfied, as, otherwise, the results obtained will be quite illusory.

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Paper No. 1462

THE DUTY OF WATER IN THE PACIFIC NORTHWEST

By J. C. Stevens, M. Am. Soc. C. E.

With Discussion by Messrs. Thomas H. Means, Robert E. Horton, and J. C. Stevens.

Synopsis

The duty of water for any particular locality is susceptible of analytical determination if the necessary physical characteristics of topography, soil and climate, and the crops to be grown, are known. The object of this paper is to strip this subject of some of the ornate generalities that usually characterize its discussion, and to suggest lines along which future research should be conducted in order to supply the missing data necessary to the solution of the problem.

The subject is first considered analytically. The process is illustrated by an actual example, and the correctness of the determination substantiated by results actually secured on projects where crop statistics and other data are available.

Briefly stated, the determination of the duty of water consists first in fixing the average percentage of soil moisture that should be maintained for the particular soil and crops, and then finding a quantity of irrigation water that, with the precipitation and the unavoidable losses, will maintain that amount of soil moisture.

This method makes the free soil moisture in the root zone of the plants the index of plant sufficiency. It is hoped that the suggestion will lead to additional research to determine the amount of free soil moisture

in various types of soils and for various crops, that will give the highest efficiency in agricultural practice. Some work along this line has already been done and the results of such investigations have been used in making up the various formulas and criteria presented in this paper.

Terms Used.

The net duty of water is taken to mean the quantity of water artificially delivered to the margin of the farms for irrigation. It will be expressed in depth on the land in feet or inches, which is identical with acre-feet or acre-inches per acre.

Precipitation includes rain, snow, hail, sleet, dew, and frost.

Irrigation water is the water artificially delivered to the land. It does not include precipitation.

Evaporation is the return of surface water to the atmosphere. It is considered in three phases—evaporation of precipitation, of irrigation water, and from soil storage.

Percolation is the passing of water in the soil beyond the reach of plant roots.

Surface waste is the surface run-off from irrigation water or precipitation.

Soil storage is the water held in the soil by capillary forces within the root zone of plants.

Water-table is the point of soil saturation by gravity water.

Hygroscopic moisture is the quantity of water an artificially heated soil will absorb from moist air upon cooling. It is not available for plant growth.

Capillary water is the water held in the interstices of the soil against gravity.

Gravity water is all water in the soil not hygroscopic or capillary. It will flow off by gravity in drains.

Plant consumption is the water taken by plant roots.

Free moisture is any soil water available for plant growth.

What Constitutes the Net Duty of Water.

The net duty of water is very simply conceived in two parts:

1.—The quantity of water actually consumed by plants; and,

2.—The losses incident to supplying that quantity to the plant roots.

The losses are of three kinds:

1.—Evaporation,

2.—Percolation,

3.—Surface waste.

Of these losses, percolation and surface waste are entirely under the control of the irrigator, while evaporation is partly under control. It is only necessary, therefore, to consider to what extent these losses, in the exercise of good husbandry, are necessary and unavoidable and thus may be considered a beneficial use of water.

Plant Consumption.—Every plant requires certain ideal conditions for optimum growth. These conditions involve a minimum water consumption. Any material departure from these ideal conditions is reflected in an increased water consumption. In other words, Nature tries to remedy any deficit of plant food, or energy, by an increase in transpiration.

It has been found that the water consumption of plants is influenced by environmental factors as follows:

1.—The addition of fertilizer to a fertile soil has no effect on plant consumption, but materially reduces the water consumption if added to an infertile soil.

2.—Soil moisture is without effect provided there is sufficient for plant growth.

3.—Radiation, *i e.,* intensity of sunlight, increases transpiration in almost direct proportion.

4.—Transpiration varies almost directly with the saturation deficit (*i. e.,* 1 — relative humidity).

5.—Cool-weather plants, such as wheat, barley, rye, oats, etc., use a minimum of water in cool weather and a maximum in warm weather.

6.—Warm-weather plants, such as corn, millet, rice, sorghums, etc., use a minimum of water in warm weather and a maximum in cool weather.

7.—Wind movement increases plant consumption.

8.—A lack of carbon dioxide, the presence of parasites, frequent cropping or any other deleterious influence will increase the water consumption.

By far the most extensive experiments on the water consumption of plants are those of Messrs. Briggs and Shantz* at Akron, Colo., 1911-15.

The method followed was to grow plants in suitable pots, with their surfaces sealed by wax. A record of the quantity of water supplied during growth was kept. At maturity the plants were removed and kiln-dried. The consumption is expressed in pounds of water per pound of dry matter or grain produced. Table 1 gives a summary of the results thus secured.

TABLE 1.—Plant Consumption in Pounds of Water per Pound of Kiln-Dried Matter and of Grain Obtained at Akron, Colo., 1911-12-13.

	Crop (Mean of Genus).	Pounds of Water per Pound of	
		Grain.	Whole plant.
Grains	Proso	454	293
	Millet	483	310
	Sorghum	767	322
	Corn	1 250	368
	Wheat	1 350	513
	Barley	1 400	534
	Oats	1 600	597
	Rye	2 000	685
Hay	Sweet Clover	.....	770
	Vetch	.....	794
	Red and Crimson Clover	.....	797
	Alfalfa, four varieties	.....	831
	Wheat and Brome Grass	.....	861
Tubers	Sugar Beets	.....	397
	Potatoes	.....	636
Weeds	Tumble Weed	.....	287
	Purslane	.....	292
	Buffalo Grass	.....	308
	Russian Thistle	.....	336
	Mountain Sage	.....	616
	Sunflower	.....	683
	Western Ragweed	.....	948

In applying the results obtained at Akron to any particular locality, a comparison of climatic conditions will indicate whether or not additions or reductions should be made. For forage crops, the consumption ratio must be reduced to terms of cured hay, instead of dry matter. Ordinarily, about 70% of the consumption for dry matter can be used with safety.

Making this reduction, for alfalfa and clover the water consumption is 0.42 ft. in depth per ton of cured hay produced. For the grains, wheat

* *Journal of Agricultural Research*, Vol. 3, No. 1, and *Bulletin 285*, Bureau of Plant Industry, U. S. Department of Agriculture.

will require 0.03 ft.; oats, 0.02 ft.; rye, 0.04 ft.; and barley and corn, 0.025 ft. in depth per bushel of yield.

With these data, it is possible to determine for any particular locality, crop classification and yields, the approximate quantity of water that will be required for plant growth.

The monthly rate at which water is consumed may be taken as approximately proportional to the monthly increment to temperature during the period of growth. Thus, assuming that growth begins April 1st, the total plant consumption may be distributed in proportion to the increase of monthly temperature over that of March.

Evaporation.—Having determined the quantity of water consumed in plant growth, it is necessary to determine the losses incident to supplying that quantity to the plants. The three sources of loss, *viz.*, evaporation, percolation, and surface waste, will be considered in order.

As previously stated, evaporation is under the partial control of the irrigator. A great saving can be made by irrigating in deep furrows, by cultivation and maintaining a mulch on the surface. For alfalfa hay, the water should be applied just before cutting in order to start the second crop. The old plants shade the ground and diminish the effect of wind, thus greatly reducing evaporation.

Evaporation is considered in three phases according to the origin of the water evaporated:

1.—From precipitation.
2.—From irrigation water at the time of application.
3.—From soil storage, *i. e.*, water raised by capillary action to the surface and then evaporated.

The amount of precipitation evaporated can only be fixed arbitrarily. Light summer showers may all be lost, and of heavier rains from 20 to 75% is likely to be dissipated. In duty-of-water calculations it is only practicable to consider the normal monthly precipitation. The following formula will give approximate results for temperatures above freezing:

$$E_r = \frac{T}{100}\sqrt{R} \qquad (1)$$

Where E_r = Evaporation from precipitation, in inches per month;
T = Mean monthly temperature, in degrees Fahrenheit;
R = Monthly precipitation, in inches.

The quantity of irrigation water evaporated immediately following its application will depend on the condition of the soil, nature of crops, and method of its application. When water is first applied the rate of evaporation is high, but the quantity available for evaporation rapidly diminishes as the water soaks into the soil. Once in the soil, it can only be evaporated by being raised to the surface by capillarity when the surface becomes dryer than the soil below. Experiments show that 10% of the quantity applied is a conservative figure for gently rolling or level lands. For steeper lands, from 15 to 25% may be allowed. Losses in excess of these quantities are readily prevented and cannot be considered a beneficial use of water.

Evaporation from soil storage constitutes the most important source of loss. It has been found that there is a fairly definite relation between the amount of evaporation, the moisture content, and temperature of the top soil layer. This relation is practically independent of the nature of the soil.

When the water-table is within the zone of capillary rise, there is a certain moisture content in the top layer which depends on the height of capillary rise for that particular soil, and the position of the water-table. For a soil of uniform texture, the free moisture content may, for practical purposes, be considered to vary uniformly from saturation at the water-table to zero at the limit of capillary rise. For example, consider a soil with 40% voids and of such fineness that the limit of capillary rise is 5 ft. If the water-table is 5 ft. below the surface, the free moisture content at the surface will be zero and there will be no evaporation. If the water-table should rise to 4 ft., the free moisture content of the top layer becomes one-fifth that at saturation, or 8% by volume, and evaporation soon begins.

The soil properties required for the determination of evaporation are the percentage of voids and the limit of capillary rise. The latter will vary somewhat with temperature, but will be found to have values varying from a few inches in coarse sands to 10 ft. or more in fine silts and clays. It can be determined by packing soils in long glass tubes closed at the bottom with muslin. The bottoms are then set in a shallow tank of water and the height to which the water rises is observed. It may require several months to determine this limit in the finer soils.

TABLE 2.—Owens Valley Soil Evaporation.

Period.	Average air temperature, Owens Valley.	Evaporation per month, in inches, from Tank No.:						
		1	2	3	4	5	6	7
		Average depth to ground-water, in inches.						
		15	59	54	47	35	23	16
1910								
June	73	3.2	0.2	1.9	1.8	1.6	4.2	5.3
July	78	3.6	0.9	3.7	4.1	4.2	5.6	5.7
August	77	4.3	2.1	4.6	5.2	6.7	8.0	7.3
September	70	4.4	1.5	3.8	3.5	5.3	8.4	5.7
October	59	3.7	0.9	2.3	2.3	3.5	3.9	4.0
November	48	2.6	0.4	0.9	1.2	1.4	1.5	2.1
December	42	2.4	0.2	0.6	0.6	0.6	0.4	1.0
1911								
January	42	2.5	0.2	0.4	0.3	0.4	0.3	0.6
February	37	3.2	0.1	0.5	0.1	0.2	0.4	0.7
March	50	3.1	0.1	0.5	0.1	0.2	0.7	1.5
April	54	3.3	0.3	1.1	1.1	1.7	3.2	3.7
May	59	3.6	1.1	2.2	3.0	4.7	6.0	5.6
The year	57.4	39.9	8.0	22.5	23.3	30.5	42.6	43.2

The results secured by Charles H. Lee,* M. Am. Soc. C. E., in Owens Valley, California, constitute the only data on soil evaporation extending over an entire year. His results also include the transpiration of salt grass. The loss was measured from buried tanks in which the water-table was maintained at various levels. In Table 2 is given a summary of his results.

In the absence of more complete information, the following formula for soil evaporation is presented. The only claim for it is that it approximately fits the results of Lee's experiments in Owens Valley.

$$E_g = 0.18 \left(1 - \frac{d}{H}\right)(T - 32)\, K \qquad (2)$$

in which

E_g = Evaporation from soil, in inches per month;
d = Depth to the water-table;
H = Height of capillary rise;
T = Mean monthly temperature;
K = Factor having a value of unity for the Owens Valley experiments.

* *Water Supply Paper No. 294*, U. S. Geological Survey.

The results obtained by R. B. Sleight,* Jun. Am. Soc. C. E., at the Denver laboratories, indicate that this formula would be approximately applicable to them if the results covered an entire year. Individual months are likely to be in error where air temperatures are used, but in the entire year such errors are compensating.

The following shows the agreement between the evaporation by the formula and that measured in Owens Valley for the year from June, 1910, to May, 1911, inclusive:

Tank No.	Observed.	By Formula.	Percentage of Error.
1	39.9	46.1	+ 14
2	8.0	17.3	+ 116
3	22.5	20.3	— 10
4	23.3	25.0	+ 7
5	30.5	31.9	+ 4
6	42.6	40.3	— 5
7	43.2	45.8	+ 6

The soil in Tanks Nos. 1 and 2 was stratified sand and clay. In the other tanks the soil was of uniform texture, a "mixture of clay, soil and fine sand". Tank No. 1 was covered with 30 in. of clean river sand and the water-table was always in this sand, while the others were covered with the original salt-grass sod. This explains the error of the formula as applied to Tanks Nos. 1 and 2.

Where ground-water is not in evidence, the formula may be written:

$$E_g = \frac{0.18}{V} M (T - 32) K \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which,

V = Percentage of voids in the soil;

M = Free moisture content of the soil, in percentage by volume.

If the soil is shaded by crops or is covered with a mulch or well cultivated, a substantial reduction must be made. This is provided for by the factor K which has decreasing values as the crop growth advances. Since crop growth is largely influenced by temperature, the factor K may be expressed in terms of mean monthly temperature. The

* "Evaporation from the Surfaces of Water and River-bed Materials", *Journal of Agricultural Research*, July 30th, 1917.

following is suggested for the case of diversified farm crops well cultivated:

$$K = 1 - \frac{1}{80}(T - 32) \quad \ldots\ldots (4)$$

Percolation.—Percolation is from two sources: (1) irrigation water, and (2) precipitation.

It has been thoroughly demonstrated in practice that the quantity of irrigation water lost by percolation is entirely under the control of the irrigator, provided his supply is sufficiently elastic to permit him to adopt the proper methods of application.

In the arid regions enough percolation from both sources must occur to overbalance the evaporation from soil storage. If this takes place, there can be no accumulation of alkali in the upper soil layers. Any percolation in excess of that required to prevent alkali accumulations cannot be considered a beneficial use of water.

The amount of percolation from precipitation is governed by three factors:

1.—Topography.
2.—Character of soil and crops.
3.—Amount and character of precipitation.

In the arid regions there is comparatively little loss from this source. During snowfall, percolation is deferred until snow melting occurs and the frost is out of the ground.

The following formula may be used to give approximate results for monthly values of precipitation of more than 1 in.; for values less than 1 in., it may safely be assumed that there will be no loss:

$$P_r = \frac{1}{10}(R - 1)\,N \quad \ldots\ldots (5)$$

in which,

P_r = percolation from precipitation, in inches per month;
R = monthly precipitation, in inches.

N is a factor having the following values:

For level or gently rolling lands of sandy loam	1.0
For steeper slopes of clay soils	0.7
For very sandy or gravelly lands	1.5

Percolation from both precipitation and irrigation water continues throughout the year, as evidenced by measured outflow from drains. The calculations for duty of water are greatly simplified if percolation is assumed to occur simultaneously with the evaporation losses from soil storage. After the total amount for the year has thus been determined, it may be redistributed in accordance with the following average percentages of the total:

Month.	Percentage.	Month.	Percentage.
January	4	July	13
February	4	August	13
March	6	September	12
April	7	October	8
May	10	November	6
June	12	December	5
		The year	100

Surface Waste.—This loss is also from the same two sources—irrigation water and precipitation. That from irrigation water is entirely under the control of the irrigator, and although it is possible to eliminate all loss from this source it is not practicable to do so.

Surface waste from nearly a thousand separate tracts was measured in Idaho by Mr. Don H. Bark* during his "Duty of Water Studies" for that State. As a result of these investigations, Mr. Bark states that "under Idaho conditions, from 7½ to 12½% of the amount applied should be allowed in water right contracts" for surface waste. On level or gently rolling land, about 10% may be taken as a liberal figure. On steeper lands an allowance of 15% of the amount applied may be justified. On sandy or gravelly soils, these figures should be halved, and on clay soils they should be increased one-third.

The surface waste from precipitation is generally called run-off, and is governed by the same factors as percolation from precipitation, but in different degrees. As a guide the following formula may be used:

$$S_r = \frac{3}{10}(R - 0.5)\,F \qquad (6)$$

Where S_r = Surface waste from precipitation, in inches per month;
R = Monthly precipitation, in inches.

* Reports, State Engineer of Idaho, 1911–14.

F is a factor having the following values:

For steeper clay soils	1.5
For gently rolling loams	1.0
For rolling sandy soils	0.5
For level sandy soils	0.2

The monthly distribution of surface waste is concurrent with the supply.

Calculation of the Net Duty of Water.

In determining the duty of water for any particular set of conditions it is necessary to consider the entire year, not the irrigation season alone. Two fundamental criteria will be advanced by which the correctness of any such determination may be judged. They are:

1.—The total quantity of water received during the mean year must be such as neither to increase nor deplete the soil storage beyond a predetermined quantity that is most advantageous to plant growth.

2.—The total amount of percolation must overbalance the evaporation originating from soil storage.

In applying the first criterion, it is necessary to determine the limits between which the soil storage may be allowed to fluctuate without harm to the plants. Then fix the quantity of irrigation water to be applied such that, after adding thereto the mean precipitation and deducting therefrom the plant consumption and necessary losses, the quantity of water in soil storage remains between these limits and at the end of the year has the same value as at the beginning.

The optimum free moisture content for healthy farm crops is known to be such that the soil voids are from 40 to 60% filled with water, leaving the remaining space for air.

The minimum free moisture content is a quantity that will just maintain plant growth without wilting. The wilting point was found from more than 1 300 determinations,* to be such that the free moisture was one-half the hygroscopic moisture. The minimum free moisture content therefore may be safely taken as double that at which wilting occurs, or just equal to the hygroscopic moisture.

For diversified farm crops, it is only necessary to consider a depth of soil equal to the root zone of plants, and experience in farm drainage

* "Water Requirements of Plants", *Bulletin 285*, Bureau of Plant Industry, U. S. Department of Agriculture, 1913.

TABLE 3.—Optimum and Minimum Free Moisture in 5 Feet of Typical Soils, Expressed as Depth in Inches on the Land.

Soil.	Percentage of voids.	Optimum free moisture content.	Minimum free moisture content.
Coarse sand	36	13.0	0.5
Fine sand	38	13.7	1.8
Sandy loam	40	14.4	3.9
Fine sandy loam	45	16.2	5.3
Loam	48	17.3	6.8
Clay loam	50	18.0	8.2
Clay	52	18.7	8.5

has proven that 5 ft. should be kept free of ground-water. Therefore, computations may be safely based on the upper 5 ft. of soil.

What an optimum soil moisture of 60% of voids filled with water, and a minimum soil moisture of double that of the wilting point, mean, in inches of depths over 5 ft. of soil, are given in Table 3. The last two columns show the upper and lower limits between which the soil storage should be maintained.

Some of the coarser soils will not hold the optimum moisture content by capillarity. Hence, it is a decided waste of water to try to maintain a moisture content greater than the capillary capacity of the soil in question.

The second criterion is based on the fact that in humid regions where the percolation exceeds soil evaporation, there are no surface accumulations of alkali. On the other hand, such accumulations are always in evidence wherever the upward movement of water in the soil is not overbalanced by the downward movement.

With these two criteria as guides, it is possible to determine the quantity of water required to maintain any desired degree of soil moisture, which, in effect, constitutes the net duty of water. This quantity is that required to balance the receipts and disbursements of water on the land during the mean year.

The physical factors that must be known in advance are:

1.—Soil characteristics as to porosity, height of capillary rise, capillary capacity, depth and uniformity of texture.

2.—Topography.

3.—Mean monthly and mean annual temperature.

4.—Mean monthly and mean annual precipitation.

5.—Probable crops and yields.

In making the calculations we may first arrive at approximate values by using the mean annual climatic factors.

Let R = Mean annual precipitation;
I = Irrigation water to be applied (net duty);
C = Total annual plant consumption;
P_i and P_r = Total annual percolation loss from irrigation water and from precipitation, respectively;
S_i and S_r = Total annual surface waste from irrigation water and from precipitation, respectively;
E_i, E_r, E_g = Evaporation from irrigation water, precipitation, and soil storage, respectively.

Balancing the total water receipts and disbursements on the land, we may write:

$$I + R = C + (P_i + P_r) + (S_i + S_r) + (E_i + E_r + E_g) \ldots . (7)$$

In this expression, R and C are known, independent quantities, E_r, P_r, and S_r, may be found in terms of R; also E_i and S_i may be expressed in terms of I. From the second criterion, we also have:

$$P_r + P_i > E_g \ldots . (8)$$

But as E_g is itself a function of both I and R and the monthly temperatures, a tentative solution only is possible.

It will be found that the net duty, I, will be given in terms of either average soil moisture content or of the position of the water-table, which in turn is an index of the soil moisture above it.

Hence, if we can determine the most advantageous soil moisture content for the particular case in hand, we may fix I, the net duty of water.

Calculated Duty for the Boise Project, United States Reclamation Service.

The application of the foregoing principles will be illustrated by a concrete example. The Boise Project in Idaho will be used, since the duty of water there has been fairly well determined.

Plant Consumption.—The crop statistical reports of the United States Reclamation Service were averaged for the years 1914-1917, inclusive. The acreage used is that of 1917. The plant consumptions as determined at Akron, Colo., were applied without reduction for climatic differences.

As shown in Table 4, a mean plant consumption of 1.06 ft. or 12.7 in. per year was indicated. In distributing this amount it is assumed that growth begins on April 1st, and that the distribution is proportional to the increase of mean monthly temperature over that of March, as in Table 5.

TABLE 4.—Average Plant Consumption, Boise Project.

Crop.	Acres.	Yield per acre.	Plant Consumption.	
			Depth, in feet.	Acre-feet.
Alfalfa	34 090	3.9 tons	1.66	56 800
Barley	3 220	20.4 bu.	0.50	1 610
Clover	7 680	2.03 tons	0.84	6 430
Corn	1 410	37.9 bu.	0.97	1 370
Mixed hay	510	1.4 tons	0.62	320
Oats	3 210	25.5 bu.	0.48	1 540
Pasture	6 550	.....	0.50	3 280
Wheat	25 680	21.2 bu.	0.63	16 100
Total	83 350	.....	1.06	87 450

TABLE 5.—Distribution of Mean Plant Consumption.

Month	Mean monthly temperature, T.	$T - 42$	Percentage of total.	Inches.
March	42	0	....	....
April	50	8	6	0.8
May	58	18	13	1.6
June	66	24	17	2.2
July	73	31	22	2.8
August	72	30	22	2.8
September	62	20	14	1.8
October	50	8	6	0.7
The season	.....	139	100	12.7

Precipitation.—The losses from precipitation and the net amount going to soil storage are next found from Formulas (1), (3), and (4), as in Table 6.

Soil Evaporation.—This is found from Formulas (3) and (4). The soil is assumed to average 5 ft. in depth and to be of level or gently rolling sandy loam. An average soil porosity of 40% was used. The water-table was assumed to be either naturally, or kept by drainage, below the 5-ft. level.

The monthly expressions for evaporation from soil storage are thus found as in Table 7.

The last column shows the multipliers of the average monthly free soil moisture content, M, to obtain the soil evaporation for that month.

TABLE 6.—Total Loss and Amount of Soil Storage.

Month.	R	T	E_r	P_r	S_r	Total lost.	To soil storage.
January	1.9	29.3	0.4	0.1	0.4	0.9	1.0
February	1.4	33.8	0.4	0.0	0.3	0.7	0.7
March	1.4	42.2	0.5	0.0	0.3	0.8	0.6
April	1.2	50.1	0.5	0.1	0.2	0.8	0.4
May	1.3	57.6	0.7	0.0	0.2	0.9	0.4
June	0.9	66.0	0.6	0.0	0.1	0.7	0.2
July	0.2	72.8	0.2	0.0	0.0	0.2	0.0
August	0.2	71.8	0.2	0.0	0.0	0.2	0.0
September	0.4	61.9	0.4	0.0	0.0	0.4	0.0
October	1.3	50.3	0.6	0.1	0.2	0.9	0.4
November	0.8	39.6	0.3	0.0	0.1	0.5	0.4
December	1.8	32.2	0.4	0.1	0.4	0.9	0.9
Total	12.8	50.6	5.2	0.4	2.2	7.8	5.0

TABLE 7.—Monthly Evaporation from Soil Storage, E_g.

Month.	$T-32$	K	E_g
January	0.0	1.00	0.0
February	1.6	0.98	0.7 M
March	12.2	0.85	4.6 M
April	18.1	0.78	6.3 M
May	25.4	0.64	7.3 M
June	34.0	0.58	8.9 M
July	40.8	0.49	9.0 M
August	39.8	0.51	9.1 M
September	29.9	0.64	8.6 M
October	18.3	0.77	6.3 M
November	7.6	0.91	3.1 M
December	0.2	1.00	0.0
The year	17.6	0.76	5.36 M

In order definitely to fix the quantity of irrigation water to be applied, first fix the desired average quantity of soil moisture to be maintained. The distribution of moisture in the soil is anything but uniform, and there is always a movement of moisture from the wet to the dry soil by capillary action. It is only practicable, however, to consider the average monthly free moisture content in these calculations.

Now substitute in Equation (7) the values already determined, and, since P_r is small, place $P_i = E_g$, also E_i and S_i each equal 0.1 I, to obtain the net duty in terms of average soil moisture:

$$I = 2.5\ E_g + 9.5 \quad \ldots\ldots (9)$$

Now substitute for E_g twelve times its mean annual value and

$$I = 160\ M + 9.5 \quad \ldots\ldots (10)$$

From Equation (10) it is evident that to maintain an average soil moisture of 10%, or $M = 0.1$, requires 25.5 in. of irrigation water; 12½% requires 29.5 in., and 15% requires 33.5 in. These, however, are only tentative values for selecting a trial value of I.

TABLE 8.—Monthly Distribution.

Month.	Percentage of total.	I.
April	4	1.2
May	12	3.6
June	18	5.4
July	23	6.9
August	24	7.2
September	13	3.9
October	6	1.8
	100	30.0

The proper average soil moisture for the standard farm crops is a fruitful line of research on which there are few or no actual data. It is believed for the case in hand that an average soil moisture of 12 to 14% should be maintained, hence a trial value of $I = 30$ in. will be taken with the monthly distribution indicated in Table 8.

Now deduct the plant consumption and all determined losses from the sum of R and I; the remainder is the amount lost by soil evaporation and percolation of irrigation water. In the first calculation, P_i and E_g are assumed to be concurrent losses. A probable value for soil moisture for April is assumed as desirable at the beginning of the irrigation season. This percentage of soil moisture is applied to the multiplier for E_g (Table 7) in order to obtain the value of E_g for the succeeding month. Double this quantity (2 E_g) is then deducted from the remainder avail-

able for P_i and E_g, to obtain the increment or decrement to soil storage which, in turn, gives the soil moisture increment or decrement from which the average soil moisture for the next month is obtained. In this manner, by repeated trials, a value of soil moisture at the beginning of the season is found such that the total soil evaporation E_g is equal to half the remainder available.

The monthly values of soil moisture must remain within the prescribed limits of optimum and minimum values heretofore established and the algebraic sum of the increments thereto must be zero. If this

TABLE 9.—Balance Sheet, Boise Project.

Month.	Receipts.	Disbursements.					Percentage of average soil moisture.
	$I+R$, in inches.	Evaporation, $E_r+E_i+E_g$, in inches.	Percolation, P_i+P_r, in inches	Surface waste, S_r+S_i, in inches.	Plant consumption, C, in inches.	Total, in inches.	
January....	1.9	0.4	0.4	0.4	....	1.2	13.4
February..	1.4	0.5	0.35	0.3	....	1.15	13.8
March.....	1.4	1.15	0.5	0.3	....	1.95	12 9
April.....	2.4	1.4	0.65	0.3	0.8	3.15	11.7
May.....	4.9	1.95	0.8	0.6	1.6	4.95	11.8
June.....	6.3	2.15	1.0	0.6	2.2	5.95	12.2
July.....	7.2	2.0	1.05	0.7	2.8	6.55	13.1
August.....	7.4	2.1	1.05	0.7	2.8	6.65	14.3
September.	4.2	2.0	1.0	0.4	1.8	5.2	12.8
October....	3.1	1.6	0.75	0.4	0 7	3.45	12.2
November..	0.8	0.7	0.5	0.1	....	1.3	11.4
December..	1.8	0.4	0.5	0.4	....	1.3	12.2
The year..	42.8	16.35	8.55	5.2	12.7	42.8	12.7

cannot be obtained with the assumed value, a new value of I is tried until a value is found that will balance the receipts and disbursements of the soil, maintain the soil storage between the prescribed limits, and neither increase nor deplete the total soil storage during the year. After this is found the percolation loss is redistributed by months in accordance with the schedule of monthly percentages previously given, and the necessary adjustments in soil moisture are made.

Table 9 shows the balance sheet for the Boise Project so obtained; and in Fig. 1 the principal data are shown graphically.

Summarizing the various quantities, Table 10 is obtained.

TABLE 10.—Summary.

	Depth, in inches.		Percentage of total.
Receipts:			
Irrigation water	30.0	....	....
Precipitation	12.8	42.8	100
Disbursements:			
Plant consumption	12.7	....	30
Evaporation.....E_g 8.15			
E_r 5.2			
E_i 3.0	16.35	....	38
Percolation.....P_r 0.4			
P_i 8.15	8.55	...	20
Surface waste.....S_r 2.2			
S_i 3.0	5.2	....	12
Total	....	42.8	....
Total losses	30.1	....	70

Note that the total percolation loss of 8.55 in. overbalances the soil evaporation loss of 8.15 in. by 5%, and that all the other criteria outlined are complied with.

In order to judge of the consistency of these figures, a study was made of the actual use of water and crop statistics of three projects of the United States Reclamation Service for the four years 1914-17, inclusive, with the results given in Table 11.

On the Klamath Project only 48% of the total water received was lost. The soil on this project is such that the danger of waterlogging the land has led the irrigators to use a comparatively small quantity of water, about 1.0 ft. being artificially applied.

On the Boise Project the average water received was 42.5 in., of which 72% was lost. In the previous analysis, a total receipt of 42.8 in. was determined as necessary, of which 70% is lost. Of this amount 12.8 in. is precipitation and 30 in. is irrigation water. The agreement between the theoretical requirements and the average used during the four years is striking.

Mr. Bark's conclusions are that: "The duty for projects planted to diversified crops on the average clay loam soils of South Idaho should be sufficient so that 2 acre-feet can be retained on each irrigated acre" and that a waste of 12½% should be provided to individuals over and

above that amount. This makes 27 in., which is comparable with the analytically determined duty for the Boise District of 30 in.

In conclusion, it is well to remember that no hard and fast rules can be laid down, and exact formulas cannot be devised. Such an analysis

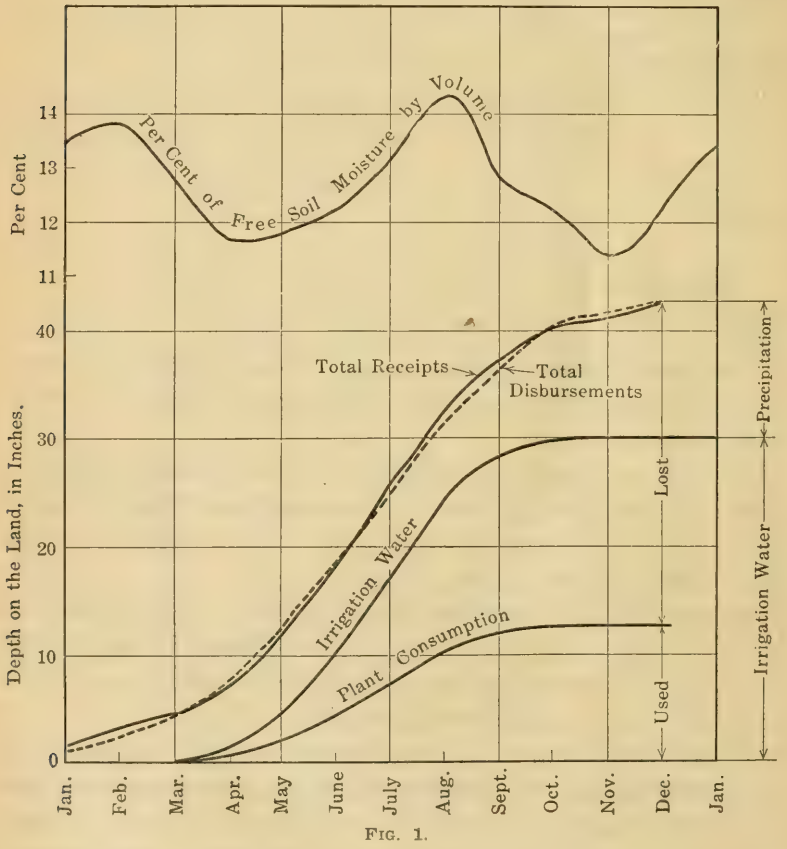

FIG. 1.

as that here outlined is a study in nice approximations in which consistency and common sense play important parts. Such analyses are necessarily based on the average, or mean climatic, year. No season exactly parallels the mean year. In dry years the quantity of irrigation water should, theoretically, be increased and during wet years it should be reduced. These yearly variations, however, can rarely be taken into

consideration in the design of irrigation works. The only practicable method is to base the design on the mean requirements and add a margin of safety for contingencies.

In the application of water to crops, there is a growing tendency to follow more scientific methods and thus eliminate unnecessary waste.

TABLE 11.—Summary of Crop Yields, Water Received and Disbursed on Three U. S. Reclamation Service Projects.

Project.	Year.	Acreage reported:	Total water received, in feet.*	Plant consumption, in feet.	Losses.	
					Depth, in feet.	Percentage of total.
Klamath	1914	23 850	1.51	0.77	0.74	49
	1915	26 810	1.41	0.69	0.72	51
	1916	29 020	1.29	0.63	0.66	51
	1917	32 510	1.13	0.73	0.40	35
	Mean..	28 050	1.34	0.70	0.64	48
Boise	1914	54 290	3.09	0.87	2.22	72
	1915	66 400	3.38	0.96	2.42	72
	1916	47 830	4.10	1.17	2.93	71
	1917	82 350	3.65	1.06	2.39	71
	Mean..	67 720	3.55	1.01	2.54	72
Truckee-Carson	1914	38 330	3.38	0.98	2.40	71
	1915	37 610	3.29	0.90	2.39	73
	1916	38 250	3.42	1.01	2.41	70
	1917	37 190	3.20	1.06	2.14	67
	Mean..	37 850	3.32	0.99	2.33	70

* Includes precipitation.

The amount of soil moisture is the best possible index of the necessity for irrigation and also of the amount to be applied. Heretofore this index has not been generally used because of the difficulty with which the amount of soil moisture could be determined. In this connection, it is perhaps fitting to add that the writer has recently designed a soil moisture indicator. This instrument is intended to be placed in the ground permanently, and indicates at the surface the relative amount of free moisture in the soil at any desired depth.

DISCUSSION

Mr. Means. THOMAS H. MEANS,* M. AM. SOC. C. E. (by letter).—This paper was read the first time with considerable interest. It covers a subject in which all irrigation engineers are interested. The duty of water has long been a subject of discussion, and its determination is one of the most difficult problems which confront the investigator. Mr. Stevens has helped to systematize the analysis, but he has gone too far in attempting to place in mathematical form so intricate a subject. A closer reading of his paper reveals many statements that are unsound. In fact, the paper on the whole is a dangerous one for the man whose knowledge of irrigation is limited to a book acquaintance with the subject. It is not the desire to discourage such enthusiasts as Mr. Stevens, but papers of this nature should not go unchallenged and allowed to become a part of scientific literature, to be quoted by the parlor irrigators who fill the periodicals with wisdom on the subject, nor to become so well recognized as to be a factor in Court decisions on irrigation questions.

There is no more complicated subject connected with the business of irrigation than that of determining the duty of water. Soil, climate, crop, the irrigator, his practices, and time, all enter the problem. Each of these can be divided into a dozen or more sub-factors and to each of them can be assigned a part in the general result. Each factor and each sub-factor may vary without regard to the others, and the final result is far from being a simple matter, capable of solution by a formula so simple as that given by Mr. Stevens.

In the writer's opinion, the greatest error in Mr. Stevens' statement of what constitutes the net duty of water is that "percolation and surface waste are entirely under the control of the irrigator." He evidently has not had an opportunity of studying in much detail the physics of soil water, nor has he studied what becomes of the water which is applied to the land. The fact that deep percolation cannot be entirely controlled and that it varies in amount with different soils is the principal reason why the use of water in sandy and gravelly areas is larger than in heavier soils. There is no reason to believe that plant consumption is any greater in sand than in loam, but it is known that in practice it is difficult to maintain good growth in a soil of coarse texture with the same quantity of water as would be sufficient in loam soil. The inability of the irrigator to apply only the quantity which can be absorbed by the soil within the zone of root action causes more deep percolation in one type of soil than in the other.

Deep percolation is usually the most important cause of the waste of irrigation water. In some California districts, carefully planned

* San Francisco, Cal.

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experiments have shown that 50% of the water received by the farmers passes below the zone of root action to such a depth that it cannot be recovered by capillarity. Improvements in methods of handling water will reduce, but not prevent, this loss. So impossible is it to prevent this deep percolation, even with the high duties now necessary in the San Joaquin Valley, that it is becoming the practice of irrigation engineers to recommend the installation of pumps to recover this underground water and thus add it to the supply. In two large districts, recently planned, it is proposed to recover, by means of wells, 20% of the quantity applied to the land, thus supplementing the original river supply.

Many other points of Mr. Stevens' paper might be discussed, but they are evident to one familiar with this subject. He uses information which properly comes from the science of plant physiology. This is a science so far out of the realm of the civil engineer that the writer asked Mr. E. E. Free, formerly of the U. S. Department of Agriculture, to review the paper and give his views on the subject. His letter follows:

"The idea of Mr. Stevens' paper is suggestive. Regarded as purely hypothetical, which is, perhaps, the way Mr. Stevens intends it, it has interest; but it is very dangerous. Mr. Stevens derives and sets forth a series of mathematical formulas for the calculation of the duty of water. These formulas rest, admittedly, not on well ascertained facts but on assumptions, guessed at from such facts as are known. However necessary or justifiable this may be in an individual problem, where the maker of the assumption knows all the local circumstances, it is very dangerous as an attempt at general conclusions. There is danger of the common error of believing the mathematics and forgetting to scrutinize the assumptions.

"Although Mr. Stevens probably would not deny that his particular assumptions are of uncertain accuracy, it is perhaps of interest to suggest specifically some of these probable inaccuracies, especially where they concern or depend on the facts and probabilities of plant physiology.

"Mr. Stevens' final formula for the duty of water, Equation (7), involves the annual rainfall; the total annual percolation losses of rainfall and of irrigation water, respectively; the similar surface wastes or run-off; the total annual evaporation losses of rainfall, irrigation water, and soil water; and a factor, C, depending on the crop grown and called the 'total annual plant consumption'. This factor C is the same as that usually called the 'water requirement' and represents the quantity of water used by the plant during the production of one weight unit of dry matter.

"The most vital inaccuracy of Mr. Stevens' discussion lies in the assumption that this factor is, as he says (page 2106), a 'known, independent' quantity. It neither is 'known' in any general sense, nor is it 'independent' of all, or even of most, of the surrounding conditions. In the calculations for the Boise Project which Mr. Stevens includes, he

Mr. Means. uses values for the factor C derived from the experiments of Drs. Briggs and Shantz, at Akron, Colo. These experiments are admirable and are sufficiently conclusive for Akron and its immediate neighborhood. It is not claimed by Drs. Briggs and Shantz, nor is it probable, that the Akron factors could be used without change for other localities unless the climatic and other conditions were substantially identical. Indeed, the entire weight of experimental evidence is against such a general validity.

"The plant consumption or water requirement (Mr. Stevens' factor C) is the quotient of two rates: (1), the rate at which water passes into the plant; and (2), the rate at which the plant produces its own substance or 'dry matter'. This second rate is simply that of growth measured in weight units. Mr. Stevens appears to consider these two rates as mutually dependent, in such fashion that their quotient which is C, is a constant, at least when the agricultural results are reasonably satisfactory. He says (page 2096), 'every plant requires certain ideal conditions for optimum growth. These conditions involve a minimum water consumption.' If this is intended as a statement of a general fact, it is not true. A complete library of plant physiology would contain some tons of discussion concerning what is the 'ideal condition of optimum growth', but the facts established and agreed to could be written on a post card. Nor does any one of the 'optimum condition complexes' proposed by various meta-physiologists necessarily, or even probably, mean a minimum water requirement. These are strong statements and will be the better for being more specific.

"Remember that the plant consumption is determined by (1) the rate of water use, and (2) the rate of growth. The water use is partly water built into the plant tissues or metabolic water and partly water evaporated from the plant or transpiration. Usually transpiration is by far the larger. The quantity of water built into tissue does depend, obviously, on growth. Transpiration, or the water passing through the plant, however, depends only slightly on the rate of growth and mainly on the evaporating power of the air. The same plant placed in warm dry air may transpire, and, therefore, 'use' as much as five times the water which it uses in warm moist air. This change in transpiration may involve less than 10% change in growth rate, provided the temperature of the two environments is the same. The writer has observed this with potato plants.

"Plant literature contains many examples of the same purport. It is true that extremely dry air may limit growth as well as affect transpiration, but transpiration may vary widely before this extreme is reached and without the variation having any proportional or important effect on the rate of growth. Furthermore, this variation of transpiration is in response to a factor entirely external to the plant, namely, the evaporating power of the air, which does not importantly affect, nor is it affected by, the rate of growth.

"This is one instance of the lack of compensating changes in water use and growth rate, and, accordingly, of lack of constancy in the factor C. Other instances of effects on one of the elements of C, without compensating effects on the other, are:

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"(1) The effect of temperature on growth rate;
(2) The effect of sunlight, and hence of photo-synthetic foods, on growth;
(3) The unbalanced effects of varietal and other variations in plant composition, especially sap concentration and acidity, on both growth rate and transpiration;
(4) The effect of the character of leaf surface, for instance, dust or water films, on transpiration;
(5) The effect of the capillary pull of the soil on the saturation deficit in the plant and thence on both transpiration and growth, but in unequal measure; and
(6) The effect of root condition, especially as modified by soil aeration, on transpiration.

"The conclusion is that the water requirement—the factor *C*—is a very complex thing. It is affected importantly by nearly every feature of the environment, and it is in the highest degree unsafe to apply to one locality numerical factors obtained in another locality. Hence the assertion that Mr. Stevens' factor *C* is neither 'known' for any area in which it has not been actually determined, nor is it 'independent' of the environmental factors of each locality. It would be scarcely less justifiable to predict the rainfall of Chicago, Ill., from the records of Washington, D. C., or New York City.

"The error of assuming greater definiteness than the present knowledge permits applies also to several other features of Mr. Stevens' calculations. Thus, Formula (1), which gives the proportion of the rainfall evaporated, is admittedly only a guess, and will be affected by wind velocity and relative humidity, neither of which appears in the formula. Formula (2), which gives the evaporation from the soil, is presented with 'the only claim for it is that it approximately fits the results of Lee's experiments in Owens Valley', obviously, a slight enough basis for general use, especially since Owens Valley is climatically unusual. Furthermore, Mr. Stevens' discussion preceding the presentation of this formula contains two serious errors. The first is that the percentage of capillary moisture in the soil is proportionate to the height above the water-table. This neglects entirely the nearly general fact that the soil texture varies between the subsoil, the soil proper, and the surface layer of soil. The second error is that the amount of evaporation from the soil is a function of its temperature and water content. This neglects the aerial factors of relative humidity and wind movement and the terranean factors of the artificial or natural creation of a dry mulch, the rate of movement of capillary water, and the diffusion of water vapor through the surface soil. The formulas for percolation and surface losses are open to uncertainties of the same kind and would require for their acceptance the presentation of many times more experimental data than is contained, or referred to, in Mr. Stevens' paper or in the generally known literature.

"Equally erroneous, though less vital to the argument, are the assumptions on page 2104 that the wilting point and optimum water content are independent of the aerial environment, and the statement on page 2096 that: '4.—Transpiration varies almost directly with the

Mr. Means. saturation deficit (*i. e.*, 1—relative humidity)'. The first assumption has been disproved by Caldwell, and the statement concerning transpiration ignores what is the greatest single factor in practice, namely, wind. It is no less an error, however unimportant practically, to regard hygroscopic, capillary, and gravity water as distinct in behavior, as Mr. Stevens apparently does on page 2095, or to say that hygroscopic water 'is not available for plant growth'. Water in the soil cannot be separated into kinds or classes and whether or not hygroscopic water is available to plants depends on the soil, the plant, the environmental conditions, and whatever definition of hygroscopic chances to have been adopted. Mr. Stevens' definition means little, since it includes the terms, artificially heated, moist air and cooling, none of which is intrinsically definite and none of which is defined.

"To summarize: Mr. Stevens has attempted a precision and generality which the facts do not justify. When Science is ready, if ever, to state a mathematical formulation of the duty of water, that formulation will be much more complex than that given by Mr. Stevens. At best, he has attempted something which is impossible at present. At worst, he is preparing to rely (and, perhaps, to have others rely) on conclusions which may be sorely misleading if applied generally. There is a gentleman of the American stage who possesses the knack of sitting down on a chair which is not there. This is a dangerous accomplishment in science. One may forget some time that the chair is hypothetical; but there will be no mistaking the reality of the bump."

Mr. Horton. Robert E. Horton,* M. Am. Soc. C. E. (by letter).—Inasmuch as the term "duty of water" is used mainly in irrigation districts, and no definition thereof is given by the author, it seems worth while to call attention to a somewhat peculiar usage of this term.

The word "duty", in irrigation practice, seems to be used in the sense of "duty performed". It is measured in terms of the depth of water required to produce an agricultural crop on irrigated land. In some respects it resembles the reciprocal of the term "efficiency", the latter being the ratio of work performed to energy supplied. "Duty", however, is a much less definite term than "efficiency" in that, as usually applied, it is not a ratio of the quantity of water supplied to the quantity of crop produced, but merely the quantity of water supplied to produce a crop. Theoretically, and in some discussions, the word "duty" is applied as meaning the quantity of water which will produce the best crop on given land. For a quantitative determination of the water requirement for irrigation, it seems desirable that some more definite term should be devised and used. Messrs. Briggs and Shantz, as well as others, have used the term "water requirement ratio" to express the ratio of the weight of the water consumed by transpiration to the weight of dry matter produced by vegetative processes. The water requirement ratio varies somewhat widely for different

* Voorheesville, N. Y.

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plants or crops, and inasmuch as transpiration is one of the principal water losses involved in irrigation, it is evident that the duty of water in irrigation must vary according to the nature of the crop produced, other things being equal. There are so many variables that, as duty is ordinarily expressed, comparisons of the duty of water in irrigation from determinations for different crops in different localities have hitherto had but little meaning.

Like other branches of applied hydrology, irrigation, is passing from the era of "rule-of-thumb" methods of pioneer practitioners to a more scientific basis of procedure. It is certainly to be desired that the various elements involved loosely in the term "duty of water" should be rendered capable of more definite measurement and comparison. The author's paper is a preliminary step toward the accomplishment of such a result. As a tentative suggestion, the expression of results obtained in irrigation by means of what may be called "irrigative efficiency" seems worthy of consideration. Assume for purposes of discussion that to produce a unit weight of dry vegetable matter with a given kind of crop requires the consumption through transpiration of at least a certain minimum volume of water, then if the total weight of dry matter of the crop produced per acre is known, the total volume of water, expressed in acre-inches, required to supply the needs of transpiration can be determined from the water requirement ratio. The "irrigative efficiency" is the ratio of the minimum quantity of water required to supply the needs of transpiration to the total quantity of water supplied to the area under cultivation.

Data regarding the water requirement ratio for most crops grown on Western irrigation lands are supplied by the experiments of Briggs and Shantz, and others. As will be pointed out, these data are not altogether satisfactory for direct application, since the water requirement ratio apparently is not constant for a given drop, as many experimentors on this subject appear to have assumed. However, using the water requirement ratio as the best basis at present available for estimating the net actual water requirements of different crops, it is easily possible to determine the irrigative efficiency of a given field and crop, if the depth of the water applied is known—furthermore, the irrigative efficiency seems to afford a very much more definite and scientific basis of comparison of the results obtainable by irrigation of different crops, and on different lands, than the indefinite term "duty of water."

Given the irrigative efficiency for one crop and area, the depth of water required is readily obtained. As an example of the application of this term, consider the growth of a crop of wheat, yielding 25 bushels per acre, on irrigated land. The water requirement ratio for spring wheat, from the experiments of Briggs and Shantz, is 1 350, that

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is to say, the weight of water lost by transpiration and consumed in vegetative processes is equal to 1 350 times the weight of dry matter, including grain and plants, above ground. Using an average ratio of weight of grain to total weight of dry matter produced above ground, and reducing these figures to equivalent inches depth of water on the surface per bushel of grain, the value of this ratio becomes 0.358. In other words, the water requirement ratio is 0.358 acre-in. per bushel of wheat per acre, or 8.95 in. depth for a yield of 25 bushels per acre. If, now, the total depth of water applied is 24 in., for example, the irrigative efficiency is 0.373.

This method leads to another useful result, as it shows at once the quantity of water applied which serves no useful purpose. The actual water requirement being 8.95 in., the irrigation losses are $24 - 8.95 = 15.05$ in.

The two questions which most concern the irrigation engineer are: (1) What becomes of this waste? and (2) By what modes of procedure may the water losses be reduced, and the irrigative efficiency consequently increased?

Now, if the crop had been 20 bushels instead of 25 bushels per acre, the duty of water, as ordinarily defined, would have been the same as before with the same depth applied, but the irrigative efficiency in the latter case would have been 30 instead of 37 per cent. This may have been the result of poor farming methods, poor methods of applying the irrigation water, unfavorable weather for crop growth, or lack of under-drainage. This example illustrates the ineptness of the expression "duty of water". It involves the effect of various factors which have to do only indirectly, if at all, with the efficiency of the water applied in crop production.

The author omits one form of water loss, namely, interception, or rainfall which is evaporated without reaching the ground. This factor is of much greater importance in relation to run-off or dry land agriculture than in relation to irrigation. However, in an irrigated region, where the rainfall is, say, 10 in. during the growing season, the loss by interception may be from 1 to 2 in.

The author distributes transpiration losses in direct proportion to the mean monthly temperature minus 32 degrees. This method, while simple, does not appear to be fully justified. It is true that transpiration per unit area of plant surface is closely correlated to air temperature, but simply for the reason that transpiration is essentially an evaporative process, and evaporation is more closely correlated to temperature than to any of the other factors affecting it. The transpiration rate is more closely correlated to evaporation than to temperature, and, therefore, the latter appears to be the better basis for distribution of the monthly amounts. This is illustrated by Table 12,

which shows the correlation coefficients between the transpiration rate for various crops, and various meteorological factors, as determined by Messrs. Briggs and Shantz. Mr. Horton.

TABLE 12.

Crop.	Date, 1914.	Evaporation.	Air temperature.	Wet bulb temperature.	Radiation.	Wind velocity.
Alfalfa	Sept. 10–Oct. 20.	0.838	0.819	0.822	0.840	0.485
"	Oct. 16–Oct. 20.	0.929	0.859	0.843	0.886	0.353
"	June 18, 19, 21..	0.888	0.788	0.852	0.861	0.626
Amaranthus.	July 7-9.....	0.946	0.849	0.842	0.844	0.683
Rye.....	June 22 July 3..	0.894	0.854	0.748	0.820	0.353
Average.....		0.90	0.834	0.822	0.850	0.50

Transpiration depth in inches per month for any crop is the product of the transpiration rate per unit surface multiplied by the area of transpiring surface. The latter increases as the crop grows. The rate of growth is a close function of effective temperature. The effective temperature in relation to the growth is generally accepted by botanists as being the excess above 41° Fahr. After the plant reaches maturity, and during the ripening period, the transpiration rate decreases.

The total transpiration surface, therefore, is approximately a function of the cumulative temperature in excess of 41° Fahr., starting at the date when the crop appears above ground; however, transpiration is not exactly proportional to area of plant surface, but is less for mature than for young vegetable tissues, so that, as shown by the experiments of Messrs. Briggs and Shantz, J. W. Leather, and others, the transpiration rate per plant under constant weather conditions follows approximately a sine curve, as also does temperature. If, therefore, transpiration rate is to be expressed as a function of temperature, it should be proportional to some power of the air temperature higher than unity, since it is a product of two factors, both accidentally, or otherwise, functions of temperature.

A better method of distributing monthly transpiration loss seems to be as follows: (1) Calculate the evaporative capacity, which can be readily done from meteorological data usually available; (2) to obtain the relative transpiration loss, multiply the evaporative capacity for each month by the transpiration-evaporation ratio. Fortunately, these ratios are much alike for most ordinary crops. Numerous values of the transpiration-evaporation ratio for different crops and plants have been worked out by Messrs. Briggs and Shantz, J. W. Leather, B. E. Livingston, and others, from whose published results the requisite data for the application of this method can be obtained.

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The author has given some attention to the matter of the quantity of infiltration. It is surprising that in view of the great importance of infiltration as a factor in all matters involving the hydrologic cycle so little attention has been devoted to the subject. Aside from meager experimental data by Messrs. O'Meara and O. R. Mathews, nothing bearing directly on this subject appears to have been published in English. It seems worth while, therefore, to consider the nature of infiltration.

In the first place, infiltrated water supplies: (*a*) The ground-water which runs off in streams; (*b*) practically the entire transpiration requirement of vegetation; and (*c*) such portion of the evaporative loss from soil surfaces as is comprised in capillary evaporation. This is usually the larger portion of soil evaporation, because at other times than during rainfall the supply of water for evaporation from soil surfaces is mainly derived from previously infiltrated water through capillary uplift.

It may be inferred from one of the author's statements, and is perhaps popularly believed, that infiltration does not take place when the ground is frozen. The writer's experience is to the contrary. If a soil which is more or less sandy becomes frozen at a time when it is not thoroughly saturated, infiltration will take place through such a soil, even though it may be frozen to a depth of several feet. The writer has observed this condition repeatedly, and the concordant rise of ground-water levels. This sometimes takes place without the frost going out of the soil to any appreciable degree.

Apparently, infiltration increases rapidly with the temperature, as would be expected from experimental results of the flow of ground-water such as those of Mr. Charles S. Slichter. At any rate, the proportion of the total rainfall which is infiltrated during the early spring and late fall is very much less than the proportion which is infiltrated during hot midsummer weather. The surface run-off in the early spring and late fall is accordingly larger. It is this fact which mainly accounts for the prevalence of floods during the spring and fall, even in the absence of melting snow, whereas equal or heavier rainfall in midsummer months is mostly infiltrated and causes but little surface run-off. It is true that the difference in midsummer conditions and conditions in cooler weather, in this regard, is often attributed to evaporation. Simple calculations will show, however, that evaporation plays a less important part than infiltration.

For example, consider a three-day storm in April or early May, with a total precipitation of 4 in., and a similar three-day rain in August, the stream being at about the same initial stage in both cases, and the drainage basin sodded. In the spring rain, the total interception and soil evaporation losses during the rain amount to, perhaps,

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½ in. Of the remaining 3½ in., on a suitable area, about 1 in. will be infiltrated, leaving 2½ in., or 62.5%, to run off the surface, this being sufficient to produce severe flood conditions on many small drainage basins. In midsummer the interception and evaporation loss during the rain may amount to 1½ in. for the three-day period. Of the remaining 2½ in., at least 2 in. will be infiltrated on most classes of cultivated areas, leaving only ½ in., or 12.5%, for surface run-off, or insufficient, as a rule, to produce flood conditions.

This statement is made for illustrative purposes only, but the difference therein indicated is believed to be realized.

The author gives a number of formulas for calculating the evaporation from the soil. The formulas undoubtedly contain the correct factors, or at least they contain factors closely correlated with those which actually control the processes; however, these formulas appear to be not only empirical but arbitrary, and it is doubtful if the factors given are in their correct relations. There is room for great advance in the hydrology of soil moisture in relation to agriculture, and the author's treatment of the subject is at least suggestive, even though it may demand marked revision.

The writer uses the formula:

$$E_s = E_w\, r$$

for calculating evaporation loss from soils. E_w is the "evaporative capacity" for a water surface having at all times a temperature equal to the soil surface, and otherwise similarly disposed. The writer has given a formula* for calculating evaporation or evaporative capacity. Subsequent experience in applying this formula to measured evaporation, as determined at thirty-five, Class A, U. S. Weather Bureau Stations, indicates that it gives very reliable results.

Buckingham has shown, and the experience of others indicates, that evaporation from the surface of a soil, other things being equal, bears the following relation to evaporation from a water surface in the same environment:

$$\frac{E_s}{E_w} = r.$$

where r is the "saturation ratio", or moisture content of the soil, expressed as a ratio to its moisture content when fully saturated or "imbibed". B. A. Keen concludes that evaporation from the soil is not directly proportional to the saturation ratio. His experiments, however, indicate that over the ordinary range of soil moisture conditions and for most soils, the assumption of direct proportionality will give at least approximately true results.

* *Engineering News-Record*, April 26th, 1917, p. 917.

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J. W. Leather has suggested the use of the exponential or compound interest law, which may be written

$$r = K_c e^{-bt}$$

for calculation of soil evaporation after rain or irrigation.

In this connection, it may be added that Buckingham's conclusion that soil evaporation is proportional to moisture content leads, under certain simple assumptions, directly to this formula for the evaporation ratio, $\frac{E_s}{E_w}$, during the time interval, t, after saturation.

The extensive experiments of Samuel Fortier, M. Am. Soc. C. E., on evaporation from soils after irrigation, give results which can be quite accurately represented by a formula of the exponential type.

The writer believes that this formula, while substantially correct and undoubtedly the simplest and best which can be used in practice, is nevertheless empirical, and not the precise or at least not the complete form of expression. This conclusion is reached from certain theoretical considerations which cannot be fully presented here for lack of space.

This paper is particularly interesting, because it represents an attempt—perhaps the first published attempt—to apply the rational water-loss method to the quantitative determination of irrigation requirements. The water-loss method of estimating the available yield of drainage basins has long been in use in a somewhat restricted form in England, and, of late, its use has progressed rapidly in the United States.

The writer has applied throughout a number of years methods of estimating stream flow closely analogous to the author's method of estimating irrigation requirements. However, the development of the same method for different purposes, and in different regions, has naturally led to different formulas for calculating the chief elements involved.

In conclusion, it appears that one of the things most needed in order that water-loss methods may be developed on a practical working basis is the determination of necessary factors and coefficients in relation to infiltration, soil moisture, and capillary flow, evaporation under the shelter of trees and crops, and transpiration rates for trees and natural vegetation other than cultivated crops. These factors, at least some of them, can be determined by an analysis of existing records of precipitation and run-off on drainage basins. When a sufficient body of carefully prepared data of this type becomes available, better estimates of water losses can undoubtedly be made.

Mr. Stevens.

J. C. Stevens,* M. Am. Soc. C. E. (by letter).—The author is grateful to Mr. Means and Mr. Free for their joint criticism of his

* Portland, Ore.

paper. He would even be more grateful if they had caught the spirit and the purpose of it. For their benefit and for that of others who may give the paper a cursory reading, the author begs to quote from the beginning of his paper: Mr. Stevens.

"The object of this paper is to strip this subject of some of the ornate generalities that usually characterize its discussion, and to suggest lines along which future research should be conducted in order to supply the missing data necessary to the solution of the problems."

and again from the closing:

"In conclusion, it is well to remember that no hard and fast rules can be laid down, and exact formulas cannot be devised. Such an analysis as that here outlined is a study in nice approximations in which consistency and common sense play important parts."

The author is fully aware of the very complex nature of every factor entering into a determination of the duty of water. They are so complex, in fact, that their individual effects have not been used, except in the most general manner, in arriving at a duty of water for any particular locality and set of conditions.

It is useless to deny that in the past the duty has been fixed largely by guess, aided of course by the experience and judgment of the guesser, and in guessing the one factor which, perhaps, has been given the greatest weight is the available water supply—a human factor, and not a physical one.

Mr. Means says that in his opinion the statement that "percolation and surface waste are entirely under the control of the irrigator" is greatly in error. He failed to bring out the author's qualifying statement which is found on page 2102:

"It has been thoroughly demonstrated in practice that the quantity of irrigation water lost by percolation is entirely under the control of the irrigator, provided his supply is sufficiently elastic to permit him to adopt the proper methods of application."

This involves the human factor to such an extent that it is well to examine briefly what practice has demonstrated. Some truly wonderful results are being secured in localities where only a very limited supply is available.

It has been often demonstrated, both by experiment and in practice, that in order to save water from percolation and surface waste on porous soils, the supply must be such as to permit quick application of small quantities at frequent intervals. It would almost seem superfluous to go into this matter further, but inasmuch as Mr. Means has branded the author's paper as a "dangerous one", a few facts will be presented in the hope of convincing those who might assume that the author's conclusions have been hastily drawn.

The Hood River District of Oregon and the Underwood District of Washington are on opposite sides of the Columbia River. Condi-

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tions of soil and climate are probably as nearly identical as it is possible to secure. In the Underwood District water is pumped to a height of 600 ft. The Hood River District has an abundant gravity supply.

In 1917, the water actually applied to certain representative tracts in the Hood River District, was measured by the State Engineer, as follows:

Tract.	Water applied (depth, in feet).	Yield.
Ia	4.86	2.80 tons clover in orchard
Ib	2.02	2.74 " " " "
Ic	6.58	3.11 " " " "
II	11.0	5.00 " " " "
III	7.13	not measured
IV	2.98	2.00 tons alfalfa in orchard

The average depth applied in the entire district of 19 000 acres was 3.3 ft.

In the Underwood District the total supply per season is limited to about 1.2 ft. in depth. Doubtless, the yields in the Underwood District are not as great as across the river, but there is no question that even with this limited supply, farming has been highly profitable. With the farmers in this district it is simply a question of 1.2 ft., or none at all. They are driven by necessity to economize to the fullest extent. In the Hood River District necessity does not so demand, and larger quantities are used. Now, this difference in water duty for the two districts is not one of physical conditions, but a difference in human endeavor pure and simple. Almost every engineer who is at all familiar with irrigation practice, can cite similar examples. The fact that large quantities of water are lost by percolation is due in most cases either to an indifference on the part of the irrigator or else to the fact that his supply is so inelastic that he could not adapt proper methods if he desired.

The saving effected by proper methods of application is well shown in Bark's experiments in Idaho.* Tanks, 6 ft. deep, were filled with the original soil from the middle of an alfalfa field by means of which the quantity of water percolating beyond the root zone was definitely measured. The tanks were cropped with the same alfalfa grown in the field.

In 1911 the same quantity of water was applied to the tanks as the farmer applied to his field, no attempt being made to alter the farmer's practice. The field and tank were irrigated seven times, applying a total of 6.6 ft. with a loss of 5.5 ft., or 84%, by percolation. The results are given in Table 13.

* Ninth Biennial Report of the State Engineer of Idaho.

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Deducting the amount of percolation and evaporation, it was found that 411 lb. of water were required to produce 1 lb. of cured hay, considerably less than was used in the calculations for the Boise District.

It was also found that only 0.15 ft. was retained in the soil at each irrigation. Accordingly, the following year the tanks were irrigated with about this quantity as often as the alfalfa appeared to need water, with the results shown in Table 14.

This shows that practically all the percolation was saved by applying small quantities frequently. In 1911 the plants snatched their

TABLE 13.—Deep Percolation from Very Gravelly Soil Cropped to Alfalfa, Rigby, Idaho.

Irrigation. 1911.	Depth applied, in feet.	Time required for absorption.	Lost by Percolation.		Time required for percolation.
			Feet.	Percentage.	
1st, May 31	1.10	2.8 hours.	0.69	62	13 days.
2d, June 14	0.71	1.5 "	0.66	93	5 "
3d, July 18	0.83	52 min.	0.79	96	9 "
4th, Aug. 1	0.64	10 "	0.51	80	8 "
5th, Aug. 9	1.09	33 "	0.97	88	5 "
6th, Aug. 31	1.14	47 "	0.94	83	6 "
7th, Sept. 24	1.09	56 "	0.95	87	3 "
Total irrigation water	6.60	.....	5.51	84	.....
Precipitation	0.31	.....	....	..	.....
Total received	6.91	.....	....	80	.....

supply from the water as it passed downward past their roots. In 1912 only the quantity was supplied that the plants required. The result was an average yield of 3.25 tons per acre in 1911, with a consumption ratio of 411 to 1, while in 1912, with a consumption of 380 lb. of water per pound of cured hay, the yield was 7.1 tons per acre.

A great saving is also effected by reducing the "length of run." By the "length of run" is meant the distance from the supply ditch to the farther end of the field. When water is supplied from a head ditch, the soil near the ditch unavoidably receives more water than that farther away. In order to irrigate properly the lower end of a field it is necessary to over-irrigate the supply end. By using a larger "irrigation head" and shortening the length of run by dividing the field by additional supply ditches a great saving may be made in the quantity of water used and in the time required for complete irrigation.

Fig. 2 shows the effect of the length of run on two fields experimented on by the U. S. Reclamation Service near Boise, Idaho, in 1910 and 1911. Fig. 2 (*b*) and (*c*) shows the saturation diagram when the

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attempt was made to saturate the ground to certain depths. As shown by Fig. 2 (*b*), to saturate the lower end of the field to a depth of 2.5 ft. required its saturation to a depth of 6.5 ft. at the upper end. If the tract had been subdivided into four equal parts by three additional supply ditches, the upper end of each tract need only have been saturated to a depth of 3.5 ft. As shown by Fig. 2 (*c*), to saturate to a depth of 4.4 ft. at the lower end required a saturation of 12.4 ft. at the upper end, with a single run, or 6.4 ft. for each tract if subdivided.

The water saved by reducing the run in Fig. 2 (*b*) was 26%, and

TABLE 14.—SAVING IN PERCOLATION LOSS EFFECTED BY FREQUENT SMALL IRRIGATIONS.

Irrigation, 1912.	Depth applied, in feet.	Lost by percolation.	Depth retained in soil.
1st, May 28	0.297	0.122	0.175
2d. June 21	0.148	Trace	0.148
3d. July 2	0.149	Trace	0.149
4th, " 6	0.148	0	0.148
5th, " 13	0.149	0	0.149
6th, " 21	0.148	0	0.148
7th, " 28	0.149	0	0.149
8th, Aug. 8	0.148	0	0.148
9th, " 13	0.149	0	0.149
10th, Sept. 2	0.148	0	0.148
Total irrigation water	1.63	0.122	1.511
Precipitation	0.71	.....	.....
Total received	2.34	.....	.....

36% in Fig. 2 (*c*). To effect these changes, larger irrigation heads for shorter periods of time are required. In Fig. 2 (*a*) is shown the duration and water consumption curves for these tracts to obtain the same degree of saturation.

For example, when Tract No. 9 was saturated in 10 hours, 2.5 acre-ft. were required, but if 30 hours were consumed with a smaller head, then 3.7 acre-ft. were required to produce the same minimum degree of saturation.

Don Bark has also included the effect of length of run in his excellent experiments. Two tracts were used: First, a strip of clover, 50 ft. wide and 2 359 ft. long; and second a strip of alfalfa, 92 ft. wide and 2 566 ft. long. The first tract was divided into seven equal parts. A constant irrigation head of 3.0 sec-ft. was turned in at the upper end to flood the strip. The time required for the water to travel to the lower end and thoroughly irrigate each division was observed. In the second case an irrigation head of 7.0 sec-ft. was used. The results are given in Table 15. This table shows that twice the water and twice

the time would be required to give this ½-mile strip the same degree of irrigation in a single run over that were it divided up into ⅛-mile runs by additional supply ditches. Mr. Stevens.

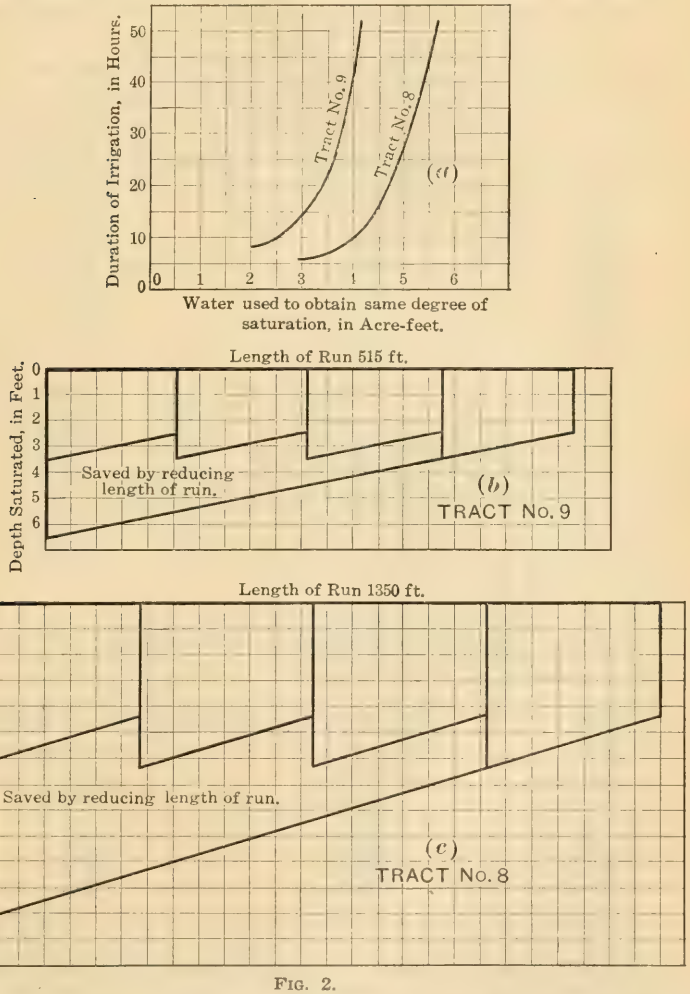

FIG. 2.

Table 16 shows similar results. With the length of run reduced to ⅛ mile, only half the water and less than half the time was required to attain the same minimum degree of irrigation over that for ½-mile runs.

The great saving that may be secured by a proper control of the human factor in irrigation practice has been demonstrated so

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thoroughly that the author feels the truth of his statement that surface waste and percolation losses are under control, and that evaporation losses are partly under the control of the irrigator, provided his supply is sufficiently elastic to permit him to adopt proper methods of application, has been demonstrated too thoroughly to merit further discussion.

TABLE 15.—Effect of Length of Run on Percolation from Clover Field Using Irrigation Head of 3.0 Second-Feet.

Length of run, in feet.	Average depth on entire tract, in feet.	Time required to irrigate entire tract had it been divided as shown, in hours.
377	0.73	8.16
674	0.82	8.77
1 011	0.88	9.40
1 348	0.93	9.90
1 685	1.11	11.80
2 022	1.26	14.50
2 359	1.47	15.67

Mr. Free has also failed to grasp the purpose and spirit of the author's use of plant transpiration as a factor in determining the duty of water. This quantity, designated as C in Equation (7), is known and independent only in the sense that it was determined prior to its inclusion in the formula. This is so evident from the treatment of this factor in the paper that it almost seems superfluous to call attention to it.

TABLE 16.—Effect of Length of Run on Alfalfa Field Using Irrigation Head of 7.0 Second-Feet.

Length of run, in feet.	Average depth on entire tract, in feet.	Time required to irrigate entire tract had it been divided as shown, in hours.
327	0.62	5.9
653	0.68	6.5
980	0.76	7.4
1 307	0.85	8.3
1 634	0.99	9.8
1 960	1.08	10.8
2 287	1.17	11.8
2 566	1.33	13.3

Any one who will study the classic experiments of Briggs and Shantz, and others, on the water requirement of plants will emerge from their study with the conviction that this factor is even more complex and more dependent on all environmental conditions than Mr. Free has pointed out. To the plant physiologist these fine distinctions are doubtless of absorbing interest, and he no doubt "views with alarm" any attempt to make practical use of the data.

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The author believes that out of all this maze of complexity some probable value for the plant consumption of standard grains, forage, and other crops can be determined sufficiently approximate for its inclusion in duty of water determinations; especially so, when it is realized that of the total quantity of water applied, only about one-third is used by the plant, the remainder being considered as necessary losses. As convincing proof of this statement one has only to consider the crops grown in humid regions where the water received is quite generally less than that applied under irrigation. Where water is supplied to crops by rainfall, the same factors of evaporation, percolation, surface run-off, and plant consumption in varying degrees, are involved, all within a less total depth, or duty, than is involved under irrigation. How, therefore, can the total plant consumption, involved factor though it may be, vary so greatly that approximate limits cannot be fixed for it within a "middle third" of the total duty? Must one who attempts to bring a little order out of chaos or to inject a bit of practical husbandry into "optimum condition complexes" be branded as an "enthusiast" and a "parlor irrigator" and his efforts as "purely hypothetical" and "very dangerous"?

The author is well aware of the looseness of the term, duty of water, as Mr. Horton points out. However, if one attempted to follow his suggested use of the term "irrigative efficiency" he must be able to fix with much greater definiteness the water requirement ratio than the author thought was justified by present knowledge of the subject.

It is to be hoped that further research will be along lines that will lead to a more general knowledge of this factor, so that its limits for various localities and crops can be better known. As a matter of pure science, an exhaustive study of all environmental and varietal influences are vital and necessary, but the agriculturalist is interested in something broader. Approximate quantities are sufficiently accurate for his purpose.

As used in the paper the term "duty of water" is intended to mean the quantity that an irrigator should have at his disposal in order to produce his crops under the average conditions of his environment. It is not the least with which the expert farmer can get along, nor the quantity the "water hog" would like to have.

The human factor enters so largely into the problem that, for a majority of localities, at least, if water could be metered to the irrigator and he was charged for what he consumes on a basis that would induce economy of use, the perplexing question of what was the proper duty for that locality would be automatically solved.

MEMOIRS OF DECEASED MEMBERS

FREDERIC PIKE STEARNS, Past-President, Am. Soc. C. E.*

DIED DECEMBER 1ST, 1919.

The life of Frederic Pike Stearns is a fine example of steady progress to the highest professional attainment, through persistent and well directed effort. It should be an inspiration to many engineers.

Frederic Pike Stearns, the son of William Henry Clark Stearns and Mary (Hobbs) Hill Stearns, was born in Calais, Me., on November 11th, 1851. He came of sturdy New England stock, having been a lineal descendant of Isaac Stearns, who came to this country from England as a member of Governor Winthrop's party and settled in Watertown, Mass., in 1630.

Mr. Stearns' early training was received in the public schools of his birthplace and was followed by a very brief business experience there. Like many another bright and active boy, he soon determined to seek the greater opportunities of the large city, and in his eighteenth year he went to Boston, Mass., to begin his education in the field of civil engineering.

He entered the office of the City Surveyor of Boston at a time when municipal engineering was in its infancy in the United States, and when the first great water-works and sanitary systems were being projected. His lack of preparation and experience would have been an unsurmountable barrier to success but for his industry and determination to fit himself to meet his task.

He began to study intensively along scientific lines, in his evenings and other spare moments, and soon attracted the attention of a notable group of engineers, then leaders of the profession in Boston, among whom were the late J. P. Kirkwood, E. S. Chesbrough, James B. Francis, Past-Presidents, Am. Soc. C. E.; Charles S. Storrow, Hon. M. Am. Soc. C. E., and Joseph P. Davis, M. Am. Soc. C. E.; and Hiram F. Mills, Hon. M. Am. Soc. C. E.

Growing in mental vigor and professional stature, Mr. Stearns soon became a leader and throughout his life was attracted by and drew about himself in his work men of strong character and alert, studious, and constructive minds, men who have played important parts in the development of municipal and sanitary engineering in the United States and abroad. Self-trained in sound and logical thinking, with an acquisitive mind, Mr. Stearns constantly broadened in vision through contact with Nature and man, calling to his assistance in his

* Memoir prepared by Leonard Metcalf, M. Am. Soc. C. E.

professional work the chemist, biologist, and bacteriologist, the geologist, architect, and landscape gardener, as the interrelation of his field and theirs became apparent to his pioneer mind—himself responding to their fresh stimulus, growing in judgment and balance, and reflecting the friendly attitude and nice courtesy of master minds to one another.

From 1872 to 1880, under the late Joseph P. Davis as Chief Engineer and Alphonse Fteley, Past-President, Am. Soc. C. E., as Resident Engineer, he served as Leveler, Assistant, and then Division Engineer in the Engineer Corps of the Boston Water-Works, having been engaged on the investigation and, later, the construction and maintenance of the additional water supply from the Sudbury River, for the City of Boston. During this period, with Mr. Fteley, he carried out an important series of hydraulic experiments on the flow of water through this conduit and over weirs, and presented the results in a paper before the Society in 1882, which won the Norman Medal, and in two subsequent papers.*

In 1880 Mr. Stearns was transferred to the Main Drainage Works of Boston, as Division Engineer in immediate charge of the tunnel under Dorchester Bay and the reservoir and other outlet works at Squantum and Moon Island, under Joseph P. Davis, City Engineer, and, later, his successor, the late Henry M. Wightman, M. Am. Soc. C. E., and with Eliot C. Clarke, M. Am. Soc. C. E., Principal Assistant Engineer. A part of his time was also devoted to the preparation of plans for the Sudbury Aqueduct crossing Farm Pond, the high-service reservoir on Fisher Hill, and a sea-wall on the Charles River Basin Embankment. From the summer of 1885 to 1886, as Executive Engineer, he had charge of the whole Main Drainage System and for a portion of this time acted as Engineering Assistant to a commission on the prevention of floods in the Valley of Stony Brook, consisting of the late Mr. James B. Francis, Mr. Eliot C. Clarke, and Clemens Herschel, Past-President, Am. Soc. C. E.

It is a noteworthy fact that these Main Drainage Works still stand as a monument of good engineering, modified only by extension to meet the growth in the district served, during this 35-year period.

In 1886 Mr. Stearns was called to the position of Chief Engineer of the Massachusetts State Board of Health. This Board

"had just been reorganized by the Legislature and, upon the recommendation of a commission of far-seeing citizens, of which the late John Quincy Adams was Chairman, had been charged with the oversight and care of the inland waters of the State and given the power to advise its municipalities and citizens upon all matters pertaining to water supply, drainage and sewerage and the prevention of the pollution of inland waters for the purpose of protecting the public health, promoting comfort and improving the living conditions of its

* See page 2137.

inhabitants. The remarkable body of able men charged with this responsibility, under the leadership of Dr. Henry P. Walcott, Chairman of the Board, and the inspiration of Mr. Hiram F. Mills, its engineer member, gathered around them the most notable group of practical scientists which had ever up to that time been associated for public health work. The work of the Board was new, no precedents existed, and the law under which the work was carried on attached no penalty in case of neglect or refusal to follow its recommendations or abide by its decisions. These decisions, in the language of those who planned this new departure in public service, were to 'look for their sanction to their own intrinsic sense and soundness', and the great influence in all matters pertaining to its work which the State Board of Health quickly acquired was due in great measure to the thoroughness and sound judgment, combined with the tact and fairness, of its chief engineer. His keen and exhaustive study of the water supplies of the State set a new standard for scientific investigations, while his tact in dealing with officials with whom he came in contact and his steadfast courage in insisting upon the right course of action secured the confidence of the public in the findings of the Board and established a means of controlling and improving the water supplies in Massachusetts which has become the basis of the practice in other States.

* * * *

"Among his notable contributions to general knowledge made in this period were his studies of stream flow and storage, establishing the principles of the economical development of the yield of water-sheds which were afterward used generally by the engineers of New England."*

During this nine-year period Mr. Stearns investigated and reported on a comprehensive sewerage system for the Mystic and Charles River Valleys, outlining plans, which were promptly approved and ordered executed by the Massachusetts Legislature; and as Chief Engineer to a Joint Board, consisting of the Massachusetts State Board of Health and the Metropolitan Park Commissioners, on the improvement of the Charles River, he recommended the construction of a dam, with a tidal lock, near Craigie Bridge, to create a fresh-water basin to be maintained at a constant level, though no work was done on it for some years. By this project, which was also exhaustively studied and reported on at a later date by John R. Freeman, M. Am. Soc. C. E., a life-long friend of Mr. Stearns, and which was subsequently executed under Hiram A. Miller, M. Am. Soc. C. E., Chief Engineer, with Mr. Stearns as Consulting Engineer, the foul estuary of the Charles River, with its tidal flats, was converted into a beautiful fresh-water basin, completely changing the character of the district and becoming one of the great recreation waterways and parks of the United States, affording badly needed relief to the congested population to the east of it.

* From Memoir of Mr. Stearns prepared by X. H. Goodnough, M. Am. Soc. C. E., for the New England Water Works Association.

In the early Nineties, Mr. Stearns came into contact with, and was much impressed by, the work on the Metropolitan Parks around Boston of the well-known landscape architect, Mr. Charles Eliot, of the firm of Olmsted, Olmsted, and Eliot, and this doubtless influenced him in his later work.

The most important work done by Mr. Stearns for the Massachusetts State Board of Health was the investigation and report on a Metropolitan Water Supply to serve the City of Boston and a large district about it, which he completed in 1895. This was the fruition of his long and close study of the water-supply problem. He, as probably no other man, sensed the need and the opportunity. He knew the state of the art and the progress of public opinion on the water-supply problem. He had helped to mould the latter along sound lines, impersonally, and with bettered service always in view, and in thus doing had won unasked the confidence of the public. Thus, naturally, there fell to him the task of building the new works.

Between 1895 and 1907, as Chief Engineer of the Metropolitan Water Board and its successor, the Metropolitan Water and Sewerage Board, he welded and inspired the engineering organization which gave effect to his ideas and built his greatest monument—the complete system of water supply for the City of Boston and nineteen surrounding municipalities, comprising great and noble dams in masonry and earth, reservoirs, aqueducts, pumping stations, and pipe systems. It may justly be said of them that they were conceived with a foresight and executed with a skill guided by the hand of Nature herself. Their worth has been demonstrated by two decades of good service. They constituted what Mr. Freeman has characterized as being at the date of their construction probably the most noteworthy series of water-works structures in the United States; foremost not altogether in size, but in perfection of detail and the embodiment of the best practice in hydraulic engineering throughout, from reservoir to pumping station.

The high standard of municipal practice set by Mr. Stearns in designing and building these Metropolitan Works has had a marked influence on important engineering structures built since in this country. The sound engineering training in investigation, design, and construction, and the object lesson of the merit of careful forethought and study in advance of construction, given by Mr. Stearns to the large number of engineers who constituted his corps and came under his influence, and who after the completion of these works scattered in different directions, to take up active work in other places the country over, where engineering work was under way, has been of far-reaching effect and of the greatest advantage to the public.

In 1900 Mr. Stearns was awarded the gold medal of the Paris Exposition for his exhibit of the Metropolitan Water Supply Works and

for work in the water-supply field. In 1905, Harvard University conferred on him the honorary degree of Master of Arts, and a year later the University of Pennsylvania further honored him with the degree of Doctor of Science.

After the completion and putting into successful operation of the Metropolitan Water Supply Works, Mr. Stearns withdrew from their active management and became the Consulting Engineer of the Board. Eminent as an engineer of wide experience and sound judgment, his advice was much sought. He limited his consulting work, however, to the problems which most interested him. Among the more important projects with which he was actively connected were the water-supply problems of New York City; Baltimore, Md.; Los Angeles, Cal.; Winnipeg, Man., Canada; Hartford, Conn.; Rochester, N. Y.; Providence, R. I.; and Worcester, Mass.; the sewerage problems of Baltimore, Md., Chicago, Ill., Pittsburgh, Pa., and elsewhere; and he was called on to pass judgment on many important dams and structures difficult of execution, built in different parts of the United States and in Mexico.

On June 24th, 1905, the late President Roosevelt appointed Mr. Stearns a member of an International Board to consider and report on plans for a canal across the Isthmus of Panama. The Board consisted of thirteen eminent engineers of whom eight members, constituting a majority, reported a plan for a sea-level canal; while a minority of five, of whom Mr. Stearns was one, reported the plan for the lock canal ultimately adopted by President Roosevelt and by the United States Government as a basis for procedure. Mr. Stearns was probably named on this Board in part for the reason of his experience in building the north dike of the Wachusett Reservoir, which reservoir was considered to be a precedent for the construction of the Bohio or Gatun Dam. Three years later, after construction had begun, the question was raised as to the stability of the Gatun Dam and other matters. President Roosevelt then appointed another Board, of which Mr. Stearns was a member, and it was this Board which accompanied President-elect Taft to the Isthmus and which revised the plans for the Gatun Dam to substantially those used in building it.*

In his work incident to the acquisition of the necessary property and rights in advance of the construction of the Metropolitan Water Supply Works, Mr. Stearns showed good judgment and fairness in dealing with the parties involved. This work carried him into the field of valuation, in which his advice was sought subsequently in a number of important cases, and led a little later to his appointment as Chairman of the Special Committee on Valuation of the Society, appointed to "formulate principles and methods for the valuation of railroad property and other public utilities."

* From information furnished the writer by Allen Hazen, M. Am. Soc. C. E., himself a member of this Board.

His character, thoroughness in detail, and fairness in dealing with difficult situations was further exemplified in the great volume of contract work which he handled without litigation, and the respect which his decisions commanded among contractors as well as engineers.

Mr. Stearns had a keen sense of his obligations to the professional societies, and took an active interest in them. He contributed many valuable papers and discussions to the publications of the Society, covering a wide field of engineering.

His more important papers were the following: "Description of Some Experiments on the Flow of Water, Made during the Construction of Works for Conveying the Water of Sudbury River to Boston";* "Disposal of Sewage in Massachusetts";† "Experiments on the Flow of Water in a 48-Inch Pipe";‡ "On the Current Meter, together with a Reason Why the Maximum Velocity of Water Flowing in Open Channels is below the Surface";‖ "The Development of Water Supplies and Water-Supply Engineering"; Presidential Address at the Annual Convention at Frontenac, Thousand Islands, N. Y., June 26th, 1906.§

He had also contributed significant discussions on other papers before the Society on algæ and the purity of water supplies; changes at the new Croton Dam; decolorization of water; going value of waterworks; hydraulic-fill dams; rainfall, flow of streams, and storage; temperature of lakes; tests of cement; the Bohio Dam; the sewerage system of Milwaukee; the Shone hydro-pneumatic system of sewerage; water power of the Falls of St. Anthony; and the hydraulic jump, in open channel flow at high velocity.

Mr. Stearns was a Past-President of the Boston Society of Civil Engineers, a Fellow of the American Academy of Arts and Sciences, and an Honorary Member of the New England Water Works Association, in the work of which he had been long and deeply interested.

In 1876, he was married to Addie C. Richardson, of Framingham, Mass., who died two years before his death. Two sons, both engineers, survive them, Herbert R. Stearns and Ralph H. Stearns, Assoc. M. Am. Soc. C. E.

Mr. Stearns was intensely human and service was his governing thought. Erect and dignified, he had a simplicity and directness of manner which attracted. Men came to him sure of his friendly interest and sound counsel. Underneath one felt instinctively his force of character, his poise and strength of purpose. As his decisions were the result of thought and careful study of details, he had the ability to

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 1.
† *Transactions*, Am. Soc. C. E., Vol. XVIII, pp. 1, 28.
‡ *Transactions*, Am. Soc. C. E., Vol. XIV, p. 1.
‖ *Transactions*, Am. Soc. C. E., Vol. XII, p. 301.
§ *Transactions*, Am. Soc. C. E., Vol. LVI, p. 451.

face strong opposition undismayed, and hold to his point. Thoroughness in the analysis of his problem and deliberate action were fundamental with him, and deservedly gave great weight to his conclusions.

The Rev. Charles E. Park, Pastor of the First Church, in Boston, which Mr. Stearns attended, has characterized him happily in the following words:

"We see him as a true patriot—a lover of his country, of his city—as one whose spirit was a public spirit, and whose time and skill and great ability were joyously employed in the careful, thoughtful, well-considered service of his fellow-citizens, both in State and Nation. * * * Constructiveness, in a spiritual as well as a literal sense, was the keynote of his life; to build up rather than to tear down. * * * He was one of those unto whom much had been given, and from whom much was required. And he met those requirements with generosity and self-forgetting fidelity. * * * It was not peoples' thanks or plaudits that he sought, but peoples' comfort. * * * Their safer, richer, easier living was dear to him."

Mr. Stearns was elected a Member of the American Society of Civil Engineers on October 2d, 1878. He also served as a Director in 1893-95, as Vice-President in 1898-99, and as President in 1906.

JAMES MURRAY AFRICA, M. Am. Soc. C. E.*

DIED SEPTEMBER 18TH, 1918.

James Murray Africa, son of the late J. Simpson Africa and Dorothea Greenland Africa, was born in Huntingdon, Pa., on April 11th, 1863. His father was a well-known surveyor of Huntingdon, who had served a term as Secretary of Internal Affairs of the State of Pennsylvania.

After completing his studies in the public schools of his native town, Mr. Africa entered Juniata College. He was graduated from Rensselaer Polytechnic Institute with high honors in 1888.

Even before his graduation from Rensselaer, Mr. Africa had been engaged in engineering work, having entered, in 1875, an engineer's office as a student. In 1882, he was appointed a member of a commission to determine the lines of Jackson, Miller, and Barree Townships in Huntingdon County, Pa. In 1882 and 1883, he was in charge of topographical and boundary surveys of the coal and ore lands of R. H. Powell and Company, and of the Huntingdon and Broad Top Mountain Railroad, in Huntingdon and Clearfield Counties, Pa. In 1884, he was elected City Engineer of Huntingdon, Pa., which office he retained for nearly thirty years, during which time he designed and constructed the sewerage system of the town, as well as its water-works.

After his graduation, Mr. Africa immediately began to engage actively in his chosen profession as a Civil and Mining Engineer and Geologist. In 1888, he designed water-works systems for Palmyra and Riverton, N. J., and was retained as Consulting Engineer by the Union Trust Company of Philadelphia, Pa., which furnished the funds for the Westmoreland County Water-Works.

In December, 1888, Mr. Africa was appointed Assistant Engineer of the Chautauqua Lake Railroad. He was afterward promoted to be Chief Engineer of that road and, in July, 1889, became also General Manager. From July, 1890, to February, 1891, he was Chief Engineer and General Manager of the Etowah Iron Company, at Cartersville, Ga., for which company he constructed 12 miles of railroad and designed and erected a mill for the concentration of manganese ore. From July to December, 1891, he was in charge of the survey of the lands of the Rockhill Iron and Coal Company, in Huntingdon County, Pa., as well as of the Robert Hare Powell coal operations in Huntingdon, Blair, and Clearfield Counties, Pa.

In 1892 and 1893, Mr. Africa designed and supervised various street improvements in Huntingdon; and, in 1894, he was appointed Chief Engineer of the Pennsylvania Midland Railroad, which position he

* Memoir prepared by the Secretary from information furnished by C. A. Sundstrom, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

held for a number of years. Among his later works were the ganister rock developments of the Harbison-Walker Company, the design and construction of the Standard Steel Works plant, at Burnham, Pa., the recent enlargement of the Pennsylvania Railroad Yards at Northumberland, Pa., and the construction of the Blair Memorial Hospital.

Mr. Africa's one hobby was Masonry. He had become affiliated with that Order at an early age, as a member of Mt. Moriah Lodge of Huntingdon, Pa., and had devoted much of his time and interest to its work. He was also a member of Standing Stone Chapter, No. 201, Royal Arch Masons, and of Huntingdon Commandery No. 65, Knights Templars.

In 1908, he was appointed Consulting and Supervising Engineer of the Masonic Homes Properties, at Elizabethtown, Pa. He superintended the construction of the buildings and laid out the grounds of these properties with the greatest interest and care, giving to the work, because of his great love for its object, the best of which he was capable, having declared that "it was one of the dearest creations of his life."

As a Civil and Mining Engineer, Mr. Africa was widely known throughout the State of Pennsylvania. He was frequently called upon as an expert in lawsuits relative to land surveys and as an appraising engineer in disputed valuations and of property acquired under right of eminent domain. His views as a Consulting Engineer were frequently sought and his opinions were held in high esteem from a legal standpoint.

Mr. Africa was prominently identified with the social, political, and civic life of his community. He was a member of the Engineers Club of Philadelphia, Pa., a Democrat in politics, and a progressive, public-spirited, and useful citizen. In writing of him at the time of his death, the Hon. George B. Orlady, of Huntingdon, states "in private life, Mr. Africa was one of our most important citizens and his untimely death will be mourned by all who ever came within the zone of his influence." In his religious affiliations, he was a consistent member of the Presbyterian Church, and at the time of his death was serving as a Trustee of that church at Huntingdon.

Mr. Africa had been in declining health for several years, suffering from hardening of the arteries and heart trouble. He had, however, kept up his business and professional interests, and for several days prior to his sudden death on September 18th, 1918, at the University Hospital, Philadelphia, Pa., had been engaged in giving expert testimony in an important lawsuit.

On June 1st, 1890, he was married to Miss Eleanor McKnight, of Reading, Pa., who, with three sons, two of whom were in the military service of the United States, during the World War, survives him.

Mr. Africa was elected a Member of the American Society of Civil Engineers, on September 6th, 1896.

HERMON CHARLES ALLEN, M. Am. Soc. C. E.*

DIED MAY 4TH, 1919.

Hermon Charles Allen was born at Coloma, Parke County, Ind., on June 2d, 1870. His primary and high school education was acquired in the county of his birth, and his technical education at Purdue University, where he studied Civil Engineering for three years as a member of the Class of 1894.

In June, 1894, Mr. Allen entered the employ of Cook and Watkins, of Boston, Mass., as Chief Draftsman, which position he retained until January, 1896. From January to June, 1896, he was in charge of the construction of the city water and electric light plant at Gas City, Ind., under the direction of James B. Nelson, a Consulting Engineer of Indianapolis, Ind. After leaving Mr. Nelson, he was engaged from June to November, 1896, as Transitman on the Chicago Division of the Cleveland, Cincinnati, Chicago and St. Louis Railway Company (the "Big Four"), but left this position to accept another with the Illinois Central Railroad Company as Transitman and Assistant Engineer on preliminary surveys, location, and construction.

On the completion of this work, in October, 1897, Mr. Allen entered the employ of the Fort Dodge and Omaha Railroad Company as Resident Engineer on location and construction. He remained with this company until March, 1899, when he returned to the Cleveland, Chicago, Cincinnati, and St. Louis Railway Company, serving from March to July, 1899, as Assistant Engineer on Maintenance of Way, and from July, 1899, to November, 1901, as Supervisor of Track, on the St. Louis Division.

Mr. Allen resigned his position with the "Big Four" to enter the office of the City Engineer of Indianapolis, Ind., as Assistant City Engineer in charge of preliminary and final estimates, street grades, etc. He remained in Indianapolis until September, 1907, when he went West to accept the position of Assistant City Engineer of Great Falls, Mont. In March, 1908, he went to Glasgow, Mont., as City Engineer, where he designed and built the electric light and water systems and made a re-survey of the city, established street grades, etc.

In June, 1909, Mr. Allen returned to Great Falls, Mont., as City Engineer, and remained in that position until 1912, when he removed to Spokane, Wash., where he engaged in private practice as a Consulting Engineer, under the firm name of Tannatt and Allen. This firm handled a number of logging, mining, and irrigation propositions in Washington and Idaho, among which the most important were the irrigation of the Hercules Ranch and the betterments to the Day Brothers Mine, at Wallace, Idaho.

* Memoir prepared by M. L. Morris, Esq., Great Falls, Mont.

Mr. Allen was engaged in the mine work when, in 1918, he was appointed Chief Engineer for the State Highway Commission of Idaho, with headquarters in Boise, which position he held practically up to the time of his death, having just severed his connection with the Commission and entered the employ of Day and Rothrock. He died of heart failure, at Sprague, Wash., on May 4th, 1919, and is survived by his wife.

With his passing, the Profession loses a member of sterling worth and a man with stamina that was unsurpassed.

Mr. Allen was elected a Member of the American Society of Civil Engineers on May 2d, 1911.

WILLIAM ARCHER, M. Am. Soc. C. E.*

DIED NOVEMBER 24TH, 1919.

William Archer was born at Penrith, England, on June 7th, 1844, and came to the United States with his parents in 1850. The Archer family settled at Cleves, Ohio, in the early Fifties, and Mr. Archer made the little suburb of Cincinnati his home for the remainder of his life. After securing his elementary education, he attended Farmers' College at College Hill, Ohio, taking a Liberal Arts Course, but specializing in all the mathematical and scientific courses available.

After spending four years as Assistant Engineer with the "Big Four" Railroad, he entered the service of the West Virginia and Ohio Railroad (now a part of the Baltimore and Ohio) in May, 1869; his active service with the Baltimore and Ohio Railroad System from that date until his death was never severed—a period of more than fifty years. He held positions, successively, as Engineer of Location and Construction, Assistant Engineer, Division Engineer, Principal Assistant Engineer, Maintenance of Way, Assistant Real Estate Agent, and Assistant Engineer in an advisory capacity, which position he held at the time of his death.

Fifty years of continuous active service, beginning with the construction of the railroad west of the Ohio River, gave Mr. Archer an almost unparalleled knowledge of the Western Lines; a wonderful memory, together with an unusual gift for assembling facts in an orderly and concise manner, enabled him to use this knowledge most effectively.

As Assistant Engineer in an advisory capacity, Mr. Archer's knowledge, together with his eagerness to assist all who came to him, made his services invaluable for the securing of historical and statistical data.

Mr. Archer was elected a Member of the American Society of Civil Engineers on March 2d, 1881; he was a Charter Member of the American Railway Engineering Association.

* Memoir prepared from information on file at the Headquarters of the Society.

THOMAS ASPINWALL, M. Am. Soc. C. E.*

DIED MARCH 2D, 1918.

Thomas Aspinwall, the son of William and Arixene Southgate (Porter) Aspinwall, was born in Brookline, Mass., on September 5th, 1853, and died in that town on March 2d, 1918.

He was educated at private schools in Boston, Mass., and at St. Mark's School, in Southboro, Mass., and was graduated with the Class of 1876 at the Massachusetts Institute of Technology.

After a short visit to England and France, Mr. Aspinwall formed a partnership with Mr. Edwin H. Lincoln, opening an office in Boston in February, 1877, under the firm name of Aspinwall and Lincoln, Civil Engineers and Surveyors. This firm is still well known in Boston, and Mr. Aspinwall was associated with it at the time of his death.

On February 3d, 1887, he married Alicia Stuart Towne, of Brookline, and is survived by his wife and two children, Lieut. Thomas Gardener Aspinwall, who served in France during the World War, and Philip Channing Aspinwall.

Mr. Aspinwall's family was one of the most distinguished in the early history of Brookline and Boston. He inherited many of the most excellent qualities of his ancestors, and was at all times a most attractive and delightful companion. He was fond of Nature, and, as a recreation from professional duties, loved to wander through the woods and fields with gun and dog.

Mr. Aspinwall's tastes were also inclined toward military affairs; between 1879 and 1882 he was a member of the First Corps of Cadets, and in 1889-91 he served as Engineer on the staff of Gen. Bridges, with the rank of Captain.

Mr. Aspinwall became a member of the Boston Society of Civil Engineers on March 17th, 1880, and was elected a Member of the American Society of Civil Engineers on May 2d, 1888.

* Memoir prepared by Desmond FitzGerald, Past-President, Am. Soc. C. E.

JOHN ABIEL ATWOOD, M. Am. Soc. C. E.*

DIED FEBRUARY 29TH, 1920.

John Abiel Atwood, the son of John Williams Atwood and Abigail (Doane) Atwood, was born at Chatham, Mass., on February 8th, 1857. He received his college education at New York University, from which he was graduated in 1878 with the degrees of B. S. and C. E.

Shortly after his graduation, Mr. Atwood accepted a position as Transitman with the Third Avenue Elevated Railroad Company of New York City, continuing as such until March, 1879. In 1879 and 1880, he served as Rodman and Leveler with the Elizabeth City and Norfolk Railroad Company (now the Norfolk Southern Railroad), and from 1881 to 1884 as Draftsman with the New York, West Shore, and Buffalo Railroad Company, with headquarters at Syracuse, N. Y.

In 1884, Mr. Atwood returned to New York City and was employed as Assistant to the Chief Engineer of the Tenth Avenue Cable Railway Company until 1886, when he accepted a position as Resident Engineer with the Chautauqua Lake Railroad Company.

From 1887 to 1889, he was engaged as Assistant Engineer with the Lake Shore and Michigan Southern Railroad Company (now part of New York Central Railroad Company).

In January, 1889, Mr. Atwood was appointed Engineer of Construction of the Pittsburgh and Lake Erie Railroad Company, which position he held until May, 1896, when he was made Chief Engineer, continuing in that capacity until his death.

He died at his residence in Beaver, Pa., on February 29th, 1920, after an attack of influenza which later developed into pneumonia with pleurisy complications. He is survived by his widow, one son, and two daughters.

During his tenure of office as Engineer of Construction and Chief Engineer, the Pittsburgh and Lake Erie Railroad grew from a single-track to a four-track railroad, which development necessitated practically the reconstruction of the entire system. Among the first engineering tasks undertaken by Mr. Atwood in his position as Chief Engineer was the relocation of the main line of the railroad, which resulted in a large reduction in grades and a marked improvement in the alignment of the road. The main line was also extended under his leadership, and several branch and tributary lines were constructed, among the latter being the Lake Erie and Eastern Railroad which is located through the heart of the City of Youngstown, Ohio. This line was replete with engineering problems of location and construction.

Large terminal, storage and classification yards, together with round-houses, and locomotive and car repair shops, were designed and built at various places along the road under Mr. Atwood's supervision, as

* Memoir prepared from information furnished by Albert R. Raymer, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

well as passenger stations and station ground layouts along the entire length of the System. In Pittsburgh, Pa., a complete terminal layout was constructed, consisting of a modern Terminal Passenger Station and General Office Building, a Terminal Freight Building, and a large fireproof Storage Warehouse.

All the bridges along the road were also rebuilt, one of the largest being the pin-connected cantilever structure over the Ohio River, at Beaver, Pa., a description of which has been presented before the Society.* On account of the many novel details embodied in its design, this bridge attracted wide attention at the time of its construction, one feature having been the trusses which are protected from damage by derailed equipment by skid girders designed and patented by Mr. Atwood.

"The true engineer is a man of vision and achievement. He derives pleasure from study and design, especially when the problem that confronts him is of an unusual type and demands original treatment. He should temper the ideal with the practical, guarding against waste and extravagance on one hand and narrowness and imperfection on the other. The possession and proper use of these qualities largely define his right to be styled an engineer. The work performed by Mr. Atwood on the Pittsburgh and Lake Erie Railroad during the quarter of a century that he served as its Chief Engineer will bear for many years to come mute testimony to the fact that he possessed in a large degree the qualities above described."

He was in the strictest sense of the term, a Christian gentleman. A man of quiet and retiring disposition, he possessed in a marked degree the ability and willingness to give due consideration to every phase of a controversy brought before him for adjudication. He commanded the highest respect of his superiors and subordinates, to all of whom he was at all times courteous. In speaking of him a few days after his death, one of his friends said that "the crowning work of his life was the development of his own high character and the highest tribute that can be paid him is the simple statement that he was 'John Atwood'".

In religion, Mr. Atwood was a Presbyterian, and had served as an Elder in the First Presbyterian Church at Beaver, Pa. He was a Director and Vice-President of the Beaver Valley Country Club and a member of the Duquesne Club of Pittsburgh, Pa. Among the technical societies to which he belonged were the Engineers' Society of Western Pennsylvania and the American Railway Engineering Association the latter of which he had served as a Director and Second Vice-President and, as Chairman of its Committee on Rails, had performed a most valued service.

Mr. Atwood was elected a Member of the American Society of Civil Engineers on January 3d, 1900.

* *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 136.

HOLLAND WILLIAMS BAKER, M. Am. Soc. C. E.*

DIED FEBRUARY 18TH, 1919.

Holland Williams Baker, the son of the late Theodore Baker and Margaret Williams Baker, natives of Connecticut and New York State, respectively, was born on September 6th, 1851, at Norwalk, Ohio, where his parents then resided. In his eleventh year, the family moved to Cleveland, Ohio, where his father was engaged in the tannery business, and where the boy attended the public schools, though somewhat irregularly, as ill health during his youth forced him to do most of his studying at home.

Mr. Baker early went into business with a brother in Chicago, Ill., but, after the great fire in that city, he returned to Cleveland where he taught school for about a year. In 1873, he entered Northwestern University as a student in Engineering, but remained only two years; he then transferred to the University of Michigan as a Junior, in the Class of 1877, with which class he was graduated, with the degree of Civil Engineer.

After his graduation in June, 1877, Mr. Baker returned first to Cleveland and then to Chicago, where he remained only until April, 1878, when he was appointed Assistant Engineer under the Missouri River Commission at St. Louis, Mo., where, engaged on the design of snagboats and other river boats, he lived until about 1883, when he went to Tacoma, Wash.

During his residence on the Pacific Coast, he was engaged in a variety of engineering work, including Government surveys, precise levels, etc., on the Columbia River. He returned to St. Louis, in 1889, as Assistant Engineer for the Mississippi River Commission, with which he was engaged until 1891, on river improvement, construction of plant, etc.

During the following ten years, Mr. Baker was in private practice and business. He spent the summer of 1897 abroad, touring the British Isles on a bicycle and going also into France.

In 1901, he went into partnership with B. H. Colby, M. Am. Soc. C. E., a classmate at the University of Michigan, under the firm name of Colby and Baker, Consulting Engineers, but after a short period the partnership was dissolved, and Mr. Baker resumed practice alone.

In July, 1905, he returned to the Government service under the United States Engineer Office, at St. Louis, on the design and construction of two large, steel hull, hydraulic dredges, *Fort Gage* and *Fort Chartres*, and many steel barges and other river plant, and was so employed until November, 1908.

During his several engagements with the United States Engineer Office and the Missouri and Mississippi River Commissions, at St.

* Memoir prepared by William S. Mitchell, M. Am. Soc. C. E.

Louis, Mr. Baker's work in design and construction, both in wood and steel, of snagboats, towboats, dredges, pile-drivers, and river vessels and plant of extremely light draft, added greatly to his reputation as a skillful engineer along such unusual lines, and he became a recognized authority in such matters, his advice therein being much sought. Subsequently, he was engaged, from 1909 to 1912, in similar work for the United States Engineer Office and the Mississippi River Commission, at Vicksburg, Miss., on the hydraulic dredge, *Waterway,* and on barges and a floating steel dry dock. In 1912, he was with the Memphis office (Mississippi River Commission), on the towboats, *Nolte* and *Reese;* from 1912 to 1914, with the Little Rock office, on the hydraulic dredges, *Taber* and *McGregor;* and from 1915 to 1916, at the Duluth office, on the steel hull dipper-dredge, *Gaillard.* It was while engaged on this last vessel, that he was obliged to retire because of his health which had been failing for several years. During the construction of the many vessels on which he was engaged, he resided for various periods at Vicksburg, Miss., Pittsburgh, Pa., Dubuque, Iowa, Green Bay, Wis., etc.

After his retirement, Mr. Baker's health continued to decline and, for the last two years of his life, he was practically confined to the house and his bed. During these latter years, his home was at Columbus, Ohio, where he died. He was buried in Bellefontaine Cemetery, St. Louis, Mo., on February 20th, 1919.

He was married, in Bloomington, Ill., on June 15th, 1898, to Miss Florine Vimont, daughter of the late John M. Vimont, of St. Louis, Mo., who survives him.

In his youth, Mr. Baker became a member of the Baptist Church with which he maintained connection until his removal to Columbus, when, with his wife, he united with the First Congregational Church, under Drs. Patton and Washington Gladden.

Throughout his entire manhood and professional career, Mr. Baker was marked for his high character and poise, and for his great sincerity, directness, and accuracy of thought. He was an excellent mathematician and an able and accomplished engineer, resourceful and accurate, and a most assiduous worker. His kindliness and simplicity, his loyalty to his work and his friends, and his perseverance and dignity, endeared him to all who knew him.

Mr. Baker was elected a Member of the American Society of Civil Engineers on May 7th, 1890. He was also a member of the St. Louis Engineers' Club.

ALONZO CLARENCE BELL, M. Am. Soc. C. E.*

DIED DECEMBER 1ST, 1918.

Alonzo Clarence Bell was born in New Orleans, La., on March 27th, 1859. He was the only son of William H. and Sophie J. Pearson Bell.

His early education was received in the schools of New Orleans, and his professional training was begun under his father, who was one of the best known engineers in Louisiana at that time.

In 1880, Mr. Bell entered the service of the United States Engineer Corps, as Assistant Engineer on river surveys and levee construction. He was in this service for four years; he next spent three years with Morgan's Louisiana and Texas Railroad and Steamship Company (now part of the Southern Pacific Railroad) designing and supervising the construction of buildings and shops. After this, he re-entered the Government service, first on lighthouse construction and, later, in charge of the construction of the Sabine Pass Jetties.

In 1890, Mr. Bell became Assistant Engineer, and, in 1893, Engineer of the Orleans Levee Board, in which positions he had full charge of the construction and maintenance of the levee system of New Orleans. From 1896 to 1900 he was City Engineer of New Orleans. From 1901 to 1907 he was in private practice. In 1907 he became Engineer for the Board of Commissioners for the Port of New Orleans, with which Board he retained connection until his death.

Mr. Bell's long experience with the problems which are peculiar to the Mississippi River rendered his judgment most valuable in all such matters. He was also especially well qualified to deal with the complex and difficult foundation problems of the alluvial basin of the Mississippi River; in addition to which, he was possessed of a broad knowledge and a high order of ability in port development and in many other branches of engineering.

Mr. Bell was a painstaking and thorough student throughout his life, and was at all times fully informed on the progress of his profession.

His extreme modesty and reserve, amounting to diffidence, made him shrink from anything which savored of self-exploitation. His rigid and unswervable integrity, and his dogged tenacity of purpose, won for him the respect of all with whom he came in contact.

He was married on July 18th, 1888, to Miss Nannie Mather, a member of one of the best known families of Louisiana. He contracted influenza in October, 1918, which, with subsequent complications, resulted in his death on December 1st, 1918. His mother and his widow survive him.

Mr. Bell was a Charter Member of the Louisiana Engineering Society and was elected a Member of the American Society of Civil Engineers on March 7th, 1900.

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

AUSTIN LORD BOWMAN, M. Am. Soc. C. E.*

DIED JUNE 3D, 1915.

Austin Lord Bowman, Chief Engineer of the Department of Bridges of New York City, and former Director of this Society, died on June 3d, 1915, after a short illness, in the prime of his vigorous and useful manhood.

He was born on November 14th, 1861, at Manchester, N. H., the second son of the Rev. George Augustus and Ernestine Lord Bowman, and was of Puritan and Pilgrim ancestry on both his father's and mother's side of the family.

He prepared for college at the public High School, Hartford, Conn., and entered Yale in 1879. He was graduated with high honors (Phi Beta Kappa) in 1883, with the degree of B. A.

Mr. Bowman's college course was marked with the same painstaking and conscientious work that characterized his professional career later, and by the determination to master whatever lines of work he undertook. Although it was necessary for him, in order to assist his finances, to engage in tutoring and other lines of student self-help, he, nevertheless, found time to participate in athletics, winning honors in boxing and wrestling. His college education was made possible largely by his own efforts, and his work and success should be an inspiration to any young man struggling for an education and an honorable professional life. In these days, when so much is heard about the study of the humanities as a supplement or foundation for an engineering course, it is interesting to note that Mr. Bowman's college education covered a B. A. degree, without the technical training thought to be so necessary at the present time.

From his work with a country surveyor during his vacations, Mr. Bowman was inclined toward engineering, and on his graduation in 1883, he at once secured a position as Rodman in a locating party for the Hartford and Harlem Railroad, between New Haven, Conn., and New York City. The Railroad Company became financially embarrassed in the late Fall, and Mr. Bowman spent the winter at home. He went to Colorado in 1884. In 1885, he worked for a short time in the office of the City Engineer of St. Paul, Minn., and left this employment to take a position as Transitman in a survey party of the Minnesota and Northwestern Railroad. This work was attended by all the hardships of a hurried railroad survey, through a swampy, heavily wooded, and uninhabited country, and with poor camp outfit.

* Memoir prepared by Albert Carr, J. A. Knighton, and Lewis D. Rights, Members, Am. Soc. C. E.

Mr. Bowman always retained a vivid recollection of the closing days of this survey, when the party subsisted on lard and blueberries.

At the close of this work, he was employed as a Transitman on the location and construction of the Chicago, St. Paul and Kansas City Railroad, and on its completion he came back to the East.

In 1887, after a short engagement on river and harbor work in New Jersey, and another short engagement on the construction of the Kings County Elevated Railroad in Brooklyn, he went to Virginia on the location and construction of the Clinch Valley Division of the Norfolk and Western Railroad. He remained with this railroad until 1890, when his chief became identified with the American Bridge and Iron Company, of Roanoke, Va. Mr. Bowman accompanied his chief as Assistant Engineer and Superintendent of Construction, beginning what, later, became his specialty in his professional life.

From 1890 to 1895, he had charge of the construction of many important bridges and other structures throughout the Southern States, and gained much practical experience which was to prove of great value to him in his later work.

In the fall of 1895 the American Bridge and Iron Company suffered financial trouble, and Mr. Bowman spent the next year in the service of the United States Government. During that period he found time to prepare and submit a competitive design for a bridge over the St. Lawrence River at Quebec, Canada, and secured second place in a world-wide competition.

In 1897, he came to New York City and opened an office as Consulting Engineer, specializing in bridge and structural steel work. He continued in private practice for ten years, and during the greater part of this period, was engaged as Consulting Bridge Engineer for the Central Railroad of New Jersey.

In December, 1907, he was appointed Consulting Engineer to the Department of Bridges of New York City, and in July, 1914, became Chief Engineer of the Department.

During his incumbency as Consulting Engineer, many important matters were submitted to him for investigation and report, notably, the question of the safety and traffic-carrying capacity of the Queensboro Bridge. His report was made the basis of the Department plan for the re-arrangement of the structure to accommodate the traffic of the dual subway system.

Under his direction, the Department carried out very elaborate and important stress-strain measurements, in connection with an investigation of the Williamsburg Bridge, the work being done with the assistance of the Bureau of Standards, S. W. Stratton, Director, at Washington, D. C. On the basis of these investigations, arrangements were made to strengthen the shore span by using an additional

bent and re-boring some of the pin-holes, so as to use larger pins. This delicate operation* was worked out to complete success.

In the report of the Department of Bridges for 1914 and 1915, there appears the following:

"At a meeting of the Commissioner and heads of Divisions of the Department of Bridges of the City of New York held on the fifth day of June, one thousand nine hundred and fifteen, the following resolution was adopted:

"*Whereas,* In the death of Austin Lord Bowman, Chief Engineer, and the technical and engineering head of this Department, we are deprived of an able adviser and a sincere friend and associate, and

"*Whereas,* It is our wish to give expression to our deep sense of loss, and to evince the respect and esteem in which Mr. Bowman was held and our keen appreciation of his lovable personality and high character; therefore be it

"*Resolved,* That all flags be displayed from the bridges of this Department at halfmast on Monday, the seventh of June, and that a copy of this resolution be transmitted to Mr. Bowman's family with the assurance of our profound regret and sincere sympathy."

Mr. Bowman served as Director of the American Society of Civil Engineers from 1905 to 1907; and, from 1909 until his death, he was Chairman of the Special Committee appointed by the Society to consider and report upon Steel Columns and Struts.

The following resolutions were approved by the Column Committee, at a meeting held at Atlantic City, N. J., on June 23d, 1915:

"*Whereas,* it has pleased Almighty Providence to call from his field of earthly labors

Austin Lord Bowman

and,

"*Whereas,* the Column Committee of the American Society of Civil Engineers was honored by his holding the position of Chairman of the Committee from the time of its appointment, and

"*Whereas,* the members of the Column Committee are desirous of expressing their sorrow on account of his loss,

"*Now be it Resolved:* That we, the members of the Column Committee deeply mourn the loss of a sincere friend, a leader among men, an eminent engineer, and a Christian gentleman.

"*Be it further Resolved:* That we desire to record our appreciation of his work in the engineering profession, but more especially on account of his untiring efforts to lead and guide the Column Com-

* *Engineering News,* Vol. 71, May 14th, 1914.

mittee in the solution of one of the great engineering problems of the present generation.

"The Column Committee of the
American Society of Civil Engineers.

"Committee:

"James H. Edwards
Charles F. Loweth
Rudolph P. Miller
Ralph Modjeski
Frank C. Osborn
George H. Pegram
Lewis D. Rights
George F. Swain
Emil Swensson
J. R. Worcester."

Coming at a time when his ability and experience could be of the greatest service to the world, Mr. Bowman's death was a distinct loss to the Engineering Profession.

He was noted for his sympathy and consideration for younger men. No young man coming to him for advice on professional or other problems went away without feeling that he had received more than money could buy.

At the time of his death, Mr. Bowman was a Member of the American Society of Mechanical Engineers, the American Society for Testing Materials, the American Railway Engineering Association, and the Association of Railway Superintendents of Bridges and Buildings. He was also a member of the Yale Club, the Engineers Club, the New York Railroad Club, a charter member of the Massachusetts Society of Mayflower Descendants, a member of the Sons of the American Revolution, and of the Society of Colonial Wars.

In 1883, he was married to Ida, daughter of Garret I. and Mary Jane Van Horne, of Jersey City, N. J., a sister of John G. Van Horne, M. Am. Soc. C. E. His wife died on May 28th, 1905. He again married, in 1907, his second wife being Eleanor, daughter of Matthew and Jane W. Heagen, of New York City, who, with their daughter, Ernestine Jane, survives him.

Mr. Bowman was elected an Associate Member of the American Society of Civil Engineers on September 7th, 1892, and a Member on December 1st, 1897.

OWEN BRAINARD, M. Am. Soc. C. E.*

DIED APRIL 2D, 1919.

Owen Brainard, well known as an architectural and civil engineer, and for many years an associate of the firm of Carrere and Hastings, died suddenly of heart stroke on April 2d, 1919, in New York City.

Owen Brainard was the son of Hubert Brainard and Cynthia Virginia Brainard, and was born on March 10th, 1865. He was descended from the first Brainard who came to the Massachusetts Bay Colony in 1642, and who was among the original proprietors of the Town of Haddam, Conn. His family has been prominent in the affairs of Middlesex County, Connecticut, through several generations, his father, grandfather, and great-grandfather having been members of the Assembly. He received his early education in the local school. Mathematics and history were his favorite studies, and he built on this foundation, by technical reading and hard study in preparation, his professional activities, also receiving instruction from professors of New York University and Stevens Institute of Technology. With an alert, inquiring, and retentive mind, he fitted himself through this study to be one of the leading men of his profession. He had the rare good fortune to choose a field of work which was a source of great satisfaction and interest to him.

Owen Brainard's father was famous in his neighborhood for his skill in masonry building, for his familiarity with mechanical appliances, and his common sense, and as a boy the son learned the practical side of his future work. In his later life, he found this early knowledge of great assistance. In a machine shop in Higganum, Conn., he learned the art of work in metals. From there he went to Philadelphia, Pa., where he lived in Germantown for a number of years.

In 1890 Mr. Brainard returned to Higganum, and there designed and built a church and several houses, one of which was of some distinction. Through these various commissions he was enabled to go abroad for study. On his return he sought New York City as a field in which to practice his profession, and his first effort was in the application of terra cotta to modern steel construction. In 1893, shortly after coming to New York, he secured the position of Engineer with the firm of Carrere and Hastings, and continued in it until his death.

Mr. Brainard's application to his work was practical. He stood for usefulness, endurance, and honesty of structure. He had a keen

* Memoir prepared by Thomas Hastings, Esq., New York City.

interest in and understanding of materials and processes of manufacture, which especially fitted him for the position of architectural engineer.

Through the large and very general practice of Carrere and Hastings, Mr. Brainard was associated with the construction of some of the most notable and important buildings in the United States. He was retained also on the general supervision and structural design of the Cuyahoga County Court House in Cleveland, Ohio, and several large churches in the West.

Mr. Brainard was the Consulting Engineer for the United States Steel Corporation in the development of several of its large towns, and was engaged during the World War in the planning of Camp Meade, and the planning and building of the Workmen's Hotels at Hog Island, Pa., for the Emergency Fleet Corporation. In later years, he had become greatly interested in large sanitation and housing problems, and, at the time of his death, he was looking forward to the study of the needs of the modern city in sanitation and planning.

On his first visit to Europe, Mr. Brainard made a study of lighthouse problems, and he was consulted by the Diamond Shoal Lighthouse Commission when the construction of that lighthouse was projected.

In 1901, Mr. Brainard was made an associate in the firm of Carrere and Hastings, and this connection continued for a number of years, until the larger development of his personal practice. However, his professional connection with the office continued until his death.

He was a member of the Engineers' Club, the Architectural League, the American Institute of Architects, and the City Planning Association. He was also a member of the Century Club, the Apawamis Club of Rye, N. Y., and the Mastigouche Fish and Game Club of Canada.

Mr. Brainard had many charming business and social qualities, and was looked up to and admired by all with whom he came in contact. Those who knew him and consulted him always retained a high regard for his clear-sighted analysis of conditions, his high standard of fair dealing, and his integrity and tactful treatment, whatever the circumstances under which the questions were submitted to him, or whatever the conclusions reached in the problem dealt with. His brevity and conciseness in expressing his conclusions gave judicial character to his technical mastery of his subject.

His recreation was in fishing and golf. He had an excellent sense of humor, and great aptness in the application of a fitting story to the question under discussion. Mr. Brainard gave much of his spare time to reading. He was an earnest worker, never neglecting his

duties, and an active participant in the work of the several professional societies to which he belonged.

On March 10th, 1909, he was married to Jean Sawyer, and made his home in New York City. He had lately built a summer home at Riverside, Conn. Besides his wife, he is survived by two brothers, Daniel and Harrison Brainard, and one sister, Ursula Brainard, who live at the old home in Higganum.

Mr. Brainard was elected a Member of the American Society of Civil Engineers on October 3d, 1906.

FREDERICK BROOKS, M. Am. Soc. C. E.*

DIED JANUARY 10TH, 1919.

Frederick Brooks, the son of Francis A. and and Frances (Butler) Brooks, was born in Boston, Mass., on July 17th, 1848, and died in that city, on January 10th, 1919.

Fred. Brooks, as he preferred to be known, was a graduate of Harvard in the Class of 1868. In college he was distinguished for proficiency in mathematics, a success which naturally led him later to enter the Massachusetts Institute of Technology, where he studied engineering from 1868 to 1870. After leaving the Institute he entered the service of Shedd and Sawyer, at that time a leading firm of Civil Engineers in Boston.

In 1881 Mr. Brooks joined the staff of the Mexican Central Railroad, and for two years was engaged in location on the Tampico Division, where he gained experience in the running of preliminary and location lines, which led to further engagements in similar work for other roads, notably for the Nashua and Lowell Railroad, with which road he remained for three years.

In 1890 Mr. Brooks entered the service of the Boston Water-Works as an Assistant Engineer, and was assigned to the South Framingham office, where he remained for eight years, working on many important problems connected with the maintenance and construction of an extensive system of water supply. When this work ended, he returned to his office in Boston, and resumed his private practice.

Fred. Brooks possessed a keen literary sense which found expression in the writing of professional papers and in the editing of the *Journal* of the Association of Engineering Societies. His papers were for the most part connected with the proposed introduction of the metric system, a project in which he was much interested, but he found time to write on "Mexican Notes," December 19th, 1883, and "Sewage Disposal at Medfield, Mass.," in July, 1888.

The Association of Engineering Societies was a wide-spread movement among the local Engineering Societies for a closer union in connection with the publication of papers. The idea originated with the Cleveland Society in 1881, and it was carried out successfully for many years, until the rapid growth of the local societies rendered it undesirable to continue the joint publication. Mr. Brooks served as editor of the *Journal* from 1905 to 1913. The duties were arduous, but he fulfilled them with ability and fidelity.

His death was accompanied by several tragic features. His health was already somewhat impaired, when he was awakened during the

* Memoir prepared by Desmond FitzGerald, Past-President, Am. Soc. C. E.

night of December 13th, 1918, by the smell of smoke in his room. After a hasty examination he found that the upper story of the house was on fire. On that floor the housekeeper was sleeping. Without thought of self, he seized a fire extinguisher and fought the fire at close range so effectively that he had almost mastered the flames when the firemen appeared and completed the work. Mr. Brooks' head, face, and hands were badly burned, and he was carried to the hospital, where he lingered until January 10th, 1919, and then passed to his long rest.

Fred. Brooks will long be remembered in Boston, where he was for so many years an interesting and prominent member of the engineering fraternity.

Mr. Brooks became a member of the Boston Society of Civil Engineers in 1874, and for the rest of his life was deeply interested in the growth and success of that Society. He was Librarian from 1878 to 1881; Director from 1890 to 1894; Vice-President from 1902 to 1904; and was elected President in 1904.

Mr. Brooks was elected a Junior of the American Society of Civil Engineers on June 7th, 1876, and a Member on January 2d, 1884.

RALPH HENRY BROWN, M. Am. Soc. C. E.*

DIED FEBRUARY 22D, 1919.

Ralph Henry Brown was born at Canfield, Ohio, on August 1st, 1860. He was the son of Richard and Thalia (Newton) Brown. His grandfather was a British officer who fought in the Battle of Waterloo, and was descended from English yeomanry ancestry. Richard Brown was born in England in 1818. He served his apprenticeship as a "draper" in London, and was engaged in business in that city and in Paris. Together with George Williams, a fellow employee, later knighted by Queen Victoria, he organized the first Young Men's Christian Association in the world. He came to the United States when twenty-one years of age, entered the employ of A. T. Stewart and Company and remained with them until about 1853, when he established himself in the dry goods business. While thus engaged, he, with Messrs. William E. Dodge and R. R. McBurney, organized in New York City the first Young Men's Christian Association in this country, and became its first Treasurer. Ralph's mother was born in Ohio, of parents who had moved there from Connecticut as early settlers in that portion of Ohio known as the Northwest Reserve.

Ralph's early education was obtained in the Collegiate School in New York City, the Grammar School at Mount Vernon, N. Y., the High School at Canfield, Ohio, of which he was a member of the first class graduated, and, later, at Wooster University, Wooster, Ohio.

From July to October, 1881, he served as Rodman and Transitman on a short extension of the Painesville and Youngstown Railroad, and for the following year was employed in the office of James M. Reno, City Engineer of Youngstown, Ohio, on city and farm surveying and in coal mine work. In 1883, he established a private practice as a Land and Mine Surveyor at Youngstown, but feeling the need of more specialized education, in the fall of that year he entered the Thayer School of Civil Engineering of Dartmouth University, at Hanover, N. H., and was graduated therefrom in March, 1885.

Immediately after his graduation, Mr. Brown entered the employ of the Boston Bridge Works as Assistant Engineer, and for the next 15 years continued his connection with this company, for 6 years as Assistant Engineer, 3 years as Engineer in Charge of the Estimating Department, and from 1894 as Chief Engineer. During his term as Chief Engineer, the Boston Bridge Works designed and built, among many structures of lesser importance, highway bridges over the Merrimack River at Newburyport, Mass., over the Kennebec

* Memoir prepared by J. R. Worcester, M. Am. Soc. C. E.

River between Fairfield and Benton, Me., over the Back River at Portland, Me., many railroad bridges, and the steel framework of many important buildings, such as armories, industrial plants, storage buildings, and hotels.

In 1900 he associated himself with Mr. A. S. Miller, Jr., in organizing the Eastern Bridge and Structural Company, of Worcester, Mass., and during the remainder of his life devoted himself exclusively to fostering and building up this company. Mr. Brown was a skillful and ingenious designer. Thoroughly grounded in the science of structures, he was intimately acquainted with the practical applications of the art, and able to devise new methods to accomplish his ends. He was always interested in economizing material and labor, and was able to effect many important savings for his clients. Though chiefly concerned with steel construction, he was alive to the uses of other materials. Largely by his ability and untiring industry, the Eastern Bridge and Structural Company has built up an enviable reputation among the users of structural steel in the eastern section of the United States.

Mr. Brown was first married in August, 1889, to Miss Ella Gertrude Gardner, of Brockton, Mass., who died in 1906, leaving a son and a daughter. He was married a second time, to Miss S. F. Palmer, only about two months before his death, which occurred on February 22d, 1919, from a sudden attack of pneumonia.

Mr. Brown was naturally retiring in his disposition and not inclined to enlarge his field of activities beyond his business associations. He was a member of the Engineers' Club of Boston, and the Worcester and the Dartmouth Clubs, of Worcester. During the last ten years of his life he took an active interest in Masonry. He was a Knight Templar and had taken nearly all the Scottish Rites Degrees. He was always a hard and persistent worker, and was just beginning to look forward to a time when he could enjoy more ease. The little time he allowed himself for recreation was spent at his summer cottage in Rutland, Mass., where he took great pleasure in hunting and fishing.

He was greatly beloved by his associates, and had few if any enemies. He had a keen sense of humor, and his hearty, bluff, sincere manner secured him a cordial welcome wherever he went.

Mr. Brown was elected a Member of the American Society of Civil Engineers on June 7th, 1899.

STEPHEN PEARSON BROWN, M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1919.

Stephen Pearson Brown, the son of Stephen O. and Mary P. Brown, was born at Dover, Me., on April 29th, 1877. After graduation from the Hotchkiss School, Lakeville, Conn., in 1896, he studied a short time in Heidelberg, Germany, and entered the Massachusetts Institute of Technology, from which he was graduated in 1900, with the degree of Bachelor of Science.

Immediately after his graduation Mr. Brown was employed in making tests of lubricating oils under mill conditions for the Washington Mills, Lawrence, Mass. From 1901 to 1903, he was junior partner of the firm of Collier and Brown, Consulting Engineers, Atlanta, Ga., having charge of the mechanical and civil engineering work in hydro-electric developments, sewerage systems, water-works systems, mine developments, and factory and electric plant designs. In 1904, he had charge of the construction of the depot at Bridgeport, Conn., under the Architect for the New York, New Haven and Hudson River Railroad Company, and, in 1905, designed reinforced concrete structures for the Columbian Fireproofing Company of New York City.

On August 1st, 1905, Mr. Brown joined the organization of the United Engineering and Contracting Company of New York City, as Engineer of Construction on the St. Mary's Park Tunnel and the Port Morris depression of the Port Morris Branch of the New York Central Railway. This company late in 1905 sent Mr. Brown to Porto Rico to investigate and report on conditions relating to municipal water-works, water supply, electric light and power from a possible water-power plant, and a sewerage system, all for the City of San Juan, Porto Rico. From 1906 to July, 1908, he had charge of the work of the same company on the Pennsylvania Railroad Company's cross-town tunnels, supervising all work west of Fifth Avenue, New York City.

The last half of 1908 Mr. Brown spent in Europe, investigating foreign practice in hydraulic and tunnel work. He returned to the United Engineering and Contracting Company in January, 1909, as Principal Assistant Engineer, and also acted as Designing Engineer for the Cuban Engineering and Contracting Company, of New York City, up to July of that year, when he returned to Europe for six months' study of tunneling methods. From 1910 to 1912, he was Chief Engineer for the Tidewater Building Company and T. B. Bryson on the Fourth Avenue Subway, New York City.

From 1912 to 1917, Mr. Brown served as Chief Engineer of the Mount Royal Tunnel and Terminal Company, Limited, and Managing

* Memoir prepared from information on file at the Headquarters of the Society.

Engineer for Mackenzie, Mann and Company, Limited, having charge of both design and construction of the terminal development in Montreal, Que., Canada, for the Canadian Northern Railway. This included a double-track tunnel more than 3 miles long, elaborate freight and passenger terminals and an elevated double-track viaduct across the lower town from the tunnel to the harbor. It involved an estimated expenditure of about $15 000 000, and $10 000 000 was actually expended.

From 1918 to June, 1919, Mr. Brown was Vice-President and General Manager of the Ford, Bacon and Davis Corporation, Engineers, and was active in the Construction Department of the International Coal Products Corporation on a Government carbo-coal and by-product plant.

Mr. Brown was a member of the following engineering societies: American Society of Mechanical Engineers, American Railway Engineering Association, Engineering Institute of Canada, and Institution of Civil Engineers (Great Britain). He was also a member of the Engineers Club, and the University Club, both of New York City.

Mr. Brown met his death at Sebec Lake, Maine, when he broke through the ice on which he was crossing to his summer home near Greeley's Landing. He directed his son, whom he had been hauling on a moose-sled, to run to the shore and down to the houses at the landing to give the alarm. The child did so and found two women, but before they could reach him, Mr. Brown had become exhausted and had sunk beneath the ice. As soon as word reached the village a party went to the landing and the body was recovered that night.

He is survived by his wife, who was Miss Edith Luce, of Boston, Mass., and two sons, his mother, and two sisters, Mrs. J. Arnold Norcross, of New Haven, Conn., and Mrs. Clarence F. Dore, of Melrose, Mass.

Mr. Brown was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1904, and a Member on November 1st, 1910.

WILLIAM LYON BROWNE, M. Am. Soc. C. E.*

DIED NOVEMBER 24TH, 1919.

William Lyon Browne, the son of William Lyon Browne and Ellen (Symonds) Browne, was born at Shrewsbury, England, on April 1st, 1876. He obtained his early education at Shrewsbury School, and after a year's study in French and German on the Continent, he spent three years acquiring a business training in mechanical work at Shropshire, England. He studied engineering at The Central Technical College of the City and Guilds of London Institute, under Professor W. C. Unwin, from 1897 to 1900.

In 1900 and 1901, Mr. Browne served under A. Ross, Chief Engineer of the Great Northern Railway (England), on the construction of the Red Bank Viaduct at Doncaster, and during 1901 and 1902, he was engaged on the design of docks and harbor works with Mr. J. J. Eady, and on surveys for tramways, etc.

In 1902, he was employed as Resident Engineer in charge of the Northern Section of the Charing Cross, Euston and Hampstead Tube Railway, in London. He was engaged on this work which consisted of eight miles of tunnel and five stations, from its inception until its completion in 1905, working under Sir Douglas Fox and Partners and W. R. Galbraith.

In August, 1905, Mr. Browne came to Canada, as representative of Sir Douglas Fox and Partners, Engineers-in-Chief of the Atlantic, Quebec, and Western Railway being constructed in the Counties of Bonaventure and Gaspé in the Province of Quebec. He remained in this position until 1907 when, in addition to continuing to act as representative of Sir Douglas Fox and Partners, he was made Chief Engineer in Canada and placed in complete charge of the construction of the Atlantic, Quebec, and Western Railway, 102 miles in length, until its completion in 1911.

Mr. Browne then accepted an appointment as Chief Engineer with the Bishop Construction Company, of Montreal, Que., Canada, which Company held a contract for the construction of part of the Quebec and Saguenay Railway, and, later, was engaged for a time as Resident Engineer for the Armstrong Whitworth Company of Canada.

From 1914 to 1916, he was employed first by the Canadian Inspection Company and, afterward, for the Imperial Government, on shell inspection.

In 1916, Mr. Browne was sent to Greece by the British Manufacturing Agents in charge of a survey for a railway for the Greek Government near Kavala, at which place he died from dysentery on November 24th, 1919.

* Memoir prepared from information supplied by John Nealor, Esq., and on file at the Headquarters of the Society.

Mr. Browne, like many English-born civil engineers, was of a pioneering disposition, as his record shows, and was intensely devoted to his adopted profession. Naturally, his first thought was safeguarding the interests of his principals, yet those who had occasion to meet and deal with him freely admitted his straightforward business integrity at all times. Socially, he was ostensibly a "home" man, devoted to his family. He had a decided preference for the sea coast, and, therefore, it is more than likely that some of the happiest years of his short life were those spent in New Carlisle, Que., Canada, on the borderland of the Gaspé Wilds.

He was extremely interested in art, was fond of open-air sports of every description, and delighted to spend his occasional leisure hours with a rod or gun. Although he had many business acquaintances, his intimate friends in Canada were comparatively few, but those who could claim such privilege deeply mourn the loss of a true, courteous, and generous companion.

Mr. Browne was married in 1903 to Ellen Winifred Nealor who, with two children, survives him.

He was a Member of the Institution of Civil Engineers of Great Britain and of the Canadian Society of Civil Engineers.

Mr. Browne was elected a Member of the American Society of Civil Engineers on January 31st, 1911.

ANDREW BRYSON, M. Am. Soc. C. E.*

DIED OCTOBER 1ST, 1918.

Andrew Bryson, the only son of Rear-Admiral Andrew Bryson and Charlotte Arnold Bryson, was born in New York City, on September 2d, 1851.

After completing his school life, Mr. Bryson began his engineering work in December, 1869, as Rodman on the Connecticut Western Railroad under the late William F. Shunk, Chief Engineer. Mr. Bryson remained on this work until its completion, having been, for the latter part of the time, in charge of a residency.

From December, 1871, to September, 1872, he was engaged on the New York and Long Branch Railroad as Leveler in charge of a locating party, and as Division Engineer preparatory to construction, which, however, was not undertaken at that time. In September and October, 1872, he served as Transitman on the location of the Rhinebeck and Connecticut and the Walkill Valley (extension) Railroads.

After severing his connection with this work, Mr. Bryson went abroad and visited England, Scotland, and France, remaining until July, 1873, when he went to Canada as Leveler on the surveys for the Missisquoi and Black River Valley Railroad, under Mr. Shunk, Chief Engineer.

In April, 1874, he was appointed Division Engineer on the Delaware, Lackawanna and Western Railroad under the late James Archbald, M. Am. Soc. C. E., Chief Engineer. Mr. Bryson retained this position until September, 1878, when he was made Engineer of the Syracuse and Binghamton Railroad, where he remained until August, 1880. During this time, he was appointed Associate Engineer on a project for a proposed bridge across the Niagara River at Lewiston, N. Y., Mr. Bryson representing the American company interested and the late Joseph Hobson, M. Am. Soc. C. E., the Canadian corporation. He also designed and superintended the construction of the docks and coal trestles of the Delaware, Lackawanna and Western Railroad at the foot of Erie Street, Buffalo, N. Y.

From August, 1880, to October, 1882, he served as Division Engineer of the Delaware, Lackawanna and Western Railroad in New York State, and located and superintended the construction of the Third Division, including much heavy work throughout the Genesee Valley.

In October, 1882, he was appointed Chief Engineer of the Hartford and Harlem Railroad and the East River and Connecticut Railroad, which were to form a proposed new line between New York City and New Britain, Conn. After the completion of this work in 1885, Mr.

* Memoir prepared from information on file at the Headquarters of the Society.

Bryson was employed as Principal Assistant Engineer on the Kings County Elevated Railway, Brooklyn, N. Y., until 1887, when he opened an office in New York City and engaged in the private practice of engineering.

In 1900, Mr. Bryson went to Reading, Pa., where he was engaged in the steel industry as President and Treasurer of the Brylgon Steel Casting Company. In 1905, in order to develop his business and to have the advantage of sea transportation facilities, Mr. Bryson moved his plant to New Castle, Del. He continued as its President until 1916, when the plant was sold to the Penn-Seaboard Steel Casting Company.

From that time until his sudden death from Spanish influenza, on October 1st, 1918, Mr. Bryson had devoted himself to community interests. He was intensely patriotic, and had been a leading spirit in the Liberty Loan and Red Cross drives in New Castle, having served as Chairman of the Men's Committee during the Second and Third Liberty Loan drives and had been engaged on plans and arrangements for the Fourth Loan when he was taken ill.

Cultured, educated, and a man of clear and sound judgment, Mr. Bryson's opinions in matters pertaining to his profession and community interests were often sought by others and were always found to be exceedingly helpful and valuable. He was thoroughly democratic in his dealings with his fellow men, demanding from them only his own standards of honesty, uprightness, and patriotism.

Mr. Bryson was a member of the St. Nicholas Club of New York City, the Loyal Legion, and President of the New Castle Branch of the Navy League. He was also a member of the Board of Trustees of the Presbyterian Church at New Castle. In connection with municipal and Federal affairs, he was a member of the New Castle Board of Trade, a commission appointed for the bonding of New Castle for street paving, the Delaware District Board for the consideration of claims and appeals in the matter of exemptions from military service, and of the Board appointed by the Government to collect information concerning the war facilities of manufacturing interests in his district.

Mr. Bryson was elected a Member of the American Society of Civil Engineers on April 1st, 1885.

ROLLA CLINTON CARPENTER, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1919.

Rolla Clinton Carpenter was born near Orion, Mich., on June 26th, 1852. His father, Charles K. Carpenter, owned an extensive farm at that place, and was the Vice-President of a railroad connecting Detroit and Bay City, which now forms a part of the Michigan Central System.

Rolla Clinton Carpenter's early education was received in the district school and in the Pontiac High School, from which he entered the Michigan Agricultural College, receiving the degree of Bachelor of Science in 1873. In 1875, he received the degree of Civil Engineer from the University of Michigan, and then returned to the Michigan Agricultural College, as an Instructor and Graduate Student, receiving the degree of Master of Science in 1876. In 1878 he was elected Professor of Mathematics and Civil Engineering at the same institution, and served in this position until 1890.

As his vacations occurred during the winter, Professor Carpenter spent a part of the time in study at other engineering colleges. At the Massachusetts Institute of Technology he studied under Professors Peabody and Lanza. His graduate study was completed at Cornell University, which conferred on him the degree of Master of Mechanical Engineering in 1888. His relation as Consulting Engineer to the Lansing Iron and Engine Company, of Lansing, Mich., placed at his disposal the facilities of a large and up-to-date manufacturing plant, which afforded opportunities for investigation exceeding those of any technical school. He was thus greatly assisted in the preparation of his thesis on the subject of "Internal Friction in Non-Condensing Engines." The late Robert H. Thurston, M. Am. Soc. C. E., then Dean of Sibley College, read a paper on this thesis before the American Society of Mechanical Engineers, and, in his discussion, showed that Professor Carpenter's investigations played an important part in changing ideas then prevalent about steam engine friction.

In 1890, Professor Carpenter was called to Cornell University by his election as Associate Professor of Mechanical Engineering in Sibley College. The mechanical laboratory work was then organized as a separate department under his direction, and, during the next five years, the equipment of the laboratories was rapidly enlarged. Professor Carpenter's intimate relation with various industrial enterprises peculiarly fitted him for the development of a course of instruction in Experimental Engineering. His success in this respect had a pronounced influence on similar courses in other engineering colleges, and on the methods of teaching other sciences. In 1895 he was promoted to a full professorship, and his title was changed to that of

* Memoir prepared by Henry S. Jacoby, Assoc. Am. Soc. C. E.

Professor of Experimental Engineering. This position he held until the time of his retirement from active service, in June, 1917.

Professor Carpenter's "Notes on Mechanical Laboratory Practice" was published in 1891. This work formed the basis of his later book on "Experimental Engineering", which has been the leading manual on the subject in the United States. The first edition of his book on heating and ventilation, entitled "Heating and Ventilating Buildings", was published in 1895. This book has had six revisions and an extensive circulation. It contains much original material from the author's own experience, and is quoted largely by later writers on the subject. He was a joint author with Professor Diederichs of a text-book on "Gas Engines." He also made numerous contributions to engineering periodicals and to the transactions of various societies, notably the American Society of Mechanical Engineers, the American Society of Civil Engineers, and the American Society of Heating and Ventilating Engineers. He was a frequent contributor to *The Sibley Journal,* of which college periodical he was an Associate Editor during the entire period of his official relation to Cornell University.

He was engaged in a diversified field of investigation and research, which included problems relating to power plants, gas engines, cement manufacture, coke manufacture, railway management, heating and ventilating, etc. He was one of the greatest patent experts in the country, and was employed by many of the leading law firms in various parts of the United States. He invented a number of pieces of laboratory apparatus, such as the Carpenter coal calorimeter, which was for many years a standard for testing the heating value of coal; the throttling and separating steam calorimeters, extensively used at present; a friction testing machine, which may be found in most of the large laboratories; and an inertia governor for the steam engine.

Professor Carpenter had been honored by appointment to various positions of distinction and trust. He was a judge of machinery and transportation at the Columbian Exposition at Chicago in 1893, at the Pan-American Exposition at Buffalo in 1901, and at the Jamestown Exposition in 1907. He was a member of the commission appointed by the Academy of Science in 1915, at the request of the President of the United States, to investigate the slides at the Panama Canal and to make such recommendations as in the judgment of the commission would improve the conditions and lessen the possibilities of slides in the future. In 1907, the Michigan Agricultural College conferred on him the degree of Doctor of Laws.

He was a member of eight of the leading engineering societies of America; during 1908-11, he was Vice-President of the American Society of Mechanical Engineers, and had served on various committees, the most important service perhaps being rendered on the Boiler Code Committee. He was President of the American Society of

Heating and Ventilating Engineers in 1898, Vice-President of the American Society of Automobile Engineers in 1910-12, and he had taken an active interest in the student branch of the American Society of Mechanical Engineers at Cornell. He was a member of Tau Beta Pi, and Sigma Xi, being President of the Alpha Chapter of the latter society in 1912-13. His relation to social and fraternal societies included membership in the Engineers' Club of New York City, the New York Railroad Club, the Town and Gown Club of Ithaca, Delta Tau Delta, and the Masonic fraternity.

Professor Carpenter's kindly manner and genial disposition made it easy for even the most timid to approach him, and he was never too busy to be considerate of any one who sought his counsel and advice. His enthusiasm in original investigation, and his capacity for hard work inspired his students to a similar attitude and effort. Outside the University his counsel was widely sought by the Government, in large engineering undertakings, and in patent legislation. His colleagues in the faculty appreciated his good judgment and his extensive knowledge of the engineering profession and of human nature. He was a pleasant companion and a loyal friend. Thus, his influence has been far-reaching both in engineering education and practice.

At the beginning of November, 1918, Professor Carpenter fell and fractured his leg, and this confined him to his home. Before this, he had made his contribution to war work by devoting himself untiringly to the development of the Liberty motor. This obliged him to travel extensively, and that is believed to have aggravated a spinal ailment due to a fall about three years before. He died on January 19th, 1919.

He was married to Miss Marion Dewey, at Greenville, Mich., on May 25th, 1876. His wife, two sons, and a daughter survive him.

Professor Carpenter was elected a Member of the American Society of Civil Engineers on April 4th, 1911.

CHARLES TITUS CHURCH, M. Am. Soc. C. E.*

DIED AUGUST 25TH, 1919.

Charles Titus Church, the son of the Rev. Samuel Clemon Church, D. D., and grandson of Capt. Samuel Church, the pioneer founder for whom Churchville was named, was born at Churchville, N. Y., on October 6th, 1834. His mother, Mary Hall (Bangs) Church, was also of early New England ancestry—a great-great-great-great granddaughter of Edward Bangs, the Pilgrim, who came from England in the *Anne* in 1623. She came from Massachusetts to make her home in Western New York in 1822.

While Dr. Samuel C. Church was pastor of the Methodist Episcopal Church, in Medina, N. Y., his son, Charles, received his academic education at the Medina Academy. He then entered Union College, from which he was graduated, with the Class of 1856, with the title of Graduate in Civil Engineering and the degree of Bachelor of Arts.

In March, 1857, Mr. Church went to Iowa and served as Rodman on the preliminary surveys for a railroad from Dubuque to Anamosa. In June, he became interested in the proposed improvements for using the water power on the Illinois side of Rock Island, but after he had designed the masonry dams, etc., the plans for this enterprise were abandoned.

In 1859, Mr. Church became a teacher in charge of a graded school at Port Byron, Ill., and, from 1861 to August, 1862, he was Superintendent of the public schools of Rock Island, Ill. From August, 1862, to July, 1864, he was engaged "in land operations" with an office at Moline, Ill.

In July, 1864, he went to Colorado in the employ of a gold mining corporation, and made topographic surveys of the mountain passes at the head-waters of the Arkansas, South Platte, and Blue Rivers and reaching over the Great Divide to the Pacific Slope. A portion of this survey was afterward used by the Pacific Railway's explorers. Mr. Church then returned to Illinois where he remained during October and November, going East in December to New York City to complete the maps of the survey, which were turned over to the Copalinshe Mining Company.

In July, 1866, he was employed as Mining Superintendent by the Champion Silver Mining Company and, in its interests, went, *via* the Isthmus of Panama, to San Francisco, Cal., and thence to Silver Peak, Nev. He made a thorough examination and a report of the Company's property at Silver Peak, but the owners decided to defer mining operations, and Mr. Church returned to New York City.

* Memoir prepared from information on file at the Headquarters of the Society.

From 1867 to 1869, he served as Agent for the American Tunnel Machine Company, directing the operation of the draftsmen and model maker. In the meantime, in November, 1868, he returned to railroad work, as Transitman on a preliminary survey for the Lake Shore Railroad from Middletown to Willimantic, Conn.

In March, 1869, he was appointed as Transitman on preliminary surveys for the New York, Ontario, and Western Railroad in Delaware and Sullivan Counties, New York State. In September of that year, he was made Assistant Engineer on Construction of the same road, which position he held for three years until the track was laid and the work completed.

In 1873, Mr. Church was engaged with the Central Railway Construction Company, of New York City, as Draftsman and Office Engineer, and from January, 1874, to May, 1875, he served as Clerk in the "eastbound" office of the Erie Railroad Company at Long Dock, Jersey City, N. J.

In May, 1875, he was appointed Assistant Engineer on the Lake Ontario Shore Railroad, in charge of construction through Oswego, N. Y. This work consisted of carrying a railroad bridge across the Oswego River and out to the new harbor.

In 1876 and 1877, Mr. Church was engaged in farming in Michigan and teaching school in Monroe County, New York State. In November, 1877, however, he went to Boston, Mass., as Office Engineer, Draftsman, etc., for Col. T. H. Dupuy, Consulting Engineer for the Boston, Hoosac Tunnel, and Western Railroad Company, which position he retained until 1880.

From August, 1880, to the fall of 1881, he was engaged on the location and construction of the Saratoga Lake Railway as Assistant Engineer and Acting Chief Engineer, respectively, and, from that time, until October, 1882, he served as Chief Engineer of the Boston Hoosac Tunnel Railroad.

From 1882 to 1886, Mr. Church was employed as Chief Engineer on the surveys, betterments, etc., for the Lebanon Springs Railroad, leaving this position to engage on location surveys for the Housatonic and Northern Railroad from the State Line, on the Boston and Albany Railroad, to Bennington and North Bennington, Vt.

In 1887, he served as Chief Engineer on the location and construction of a railroad from Derby to Botsford, Conn., in the interest of the Housatonic Railroad Company, and, in 1888 and 1889, he was engaged in the private practice of engineering, with an office at Saratoga Springs, N. Y.

From March to July, 1890, Mr. Church was employed as Assistant Engineer by the Lehigh Valley Railway Company on construction covering the crossings of the Genesee River and Oatka Creek, and from October, 1890, to October, 1892, he served as Civil Engineer for

the Retsof Mining Company on mine surveys, mapping, and railroad construction connecting the mine with the Delaware, Lackawanna, and Western Railroad, as well as with several other railroads near Caledonia, N. Y.

From October, 1892, to July, 1895, Mr. Church was employed as General Freight and Passenger Agent of the Genesee and Wyoming Valley Railroad Company, and from that time until 1899, he was engaged in private practice, surveying, mapping lands, etc.

In the meantime, his family had become residents of Geneva, N. Y., and, in 1899, Mr. Church became identified with the municipal government of that city, serving for two years as City Engineer and afterward as City Engineer and Superintendent of Public Works until 1912, when he resigned and retired from active work. He continued his residence in Geneva where he died on August 25th, 1919. He is survived by a nephew and several nieces.

Mr. Church was married, in New York City, on August 26th, 1873, to Miss Frances Ann Van Zandt, daughter of Beekman and Frances Susanna (Van Buren) Van Zandt. Mrs. Church died on October 14th, 1910.

Mr. Church was thorough and accurate in all his work and undertakings, as thorough in his reading as in other things, a reader of many books, papers, and magazines, and interested in events of the day to the very last. Hospitable, gallant, suave—something of the type of the "old-school gentleman"—with an excellent memory, keen wit, and remarkably lucid expression of his thoughts or opinions, Mr. Church was a delightfully entertaining host; but he did not care to visit his club or other places where men usually gather; he liked the quiet of his home. He was a member of the University Club, the Elks, and the Masons, but never held office in any of them. He was a Vestryman of Trinity Protestant Episcopal Church, in Geneva, N. Y., from 1894 to 1917, and for some years served as Clerk of the Vestry.

Mr. Church was elected a Member of the American Society of Civil Engineers on May 1st, 1889.

HOWARD LINCOLN COBURN, M. Am. Soc. C. E.*

DIED JUNE 18TH, 1918.

Howard Lincoln Coburn was born on January 13th, 1867, at Patten, Me., the son of America Thayer Coburn, a merchant of that town, and Annabelle (Tine) Coburn, of Lee, Me. He attended the schools of Patten and entered the Massachusetts Institute of Technology in 1883 as a member of the Class of '87.

When he had completed two years of his course in Mechanical Engineering his father met with reverses in business and Mr. Coburn promptly withdrew from college and went to work. He entered the office of Mr. Charles Carr, a mill engineer, of Boston, Mass., with whom he spent most of his time during the ensuing ten years, designing buildings and equipment for New England cotton mills.

After ten years in business, Mr. Coburn re-entered the Massachusetts Institute of Technology in 1896, and was graduated in 1898 with the degree of S. B.

Again taking up mill construction, he spent two years in the office of the late E. D. Leavitt, M. Am. Soc. C. E., of Boston, and then became connected with the firm of Lockwood, Greene and Company, with which he spent ten years, and rose to be Chief Engineer.

In 1905 he began the work which has made his name most familiar to the Engineering Profession, and to it he continued to devote his talents up to the time of his death. He joined the newly organized Ambursen Hydraulic Construction Company, as Chief Engineer. In the following years, he was responsible for the construction of more than one hundred reinforced concrete dams and hydro-electric developments. His work covered nearly every State in the Union, and included many installations in Canada and the West Indies. Some of his biggest and best known works are the Guayabal Dam, for the Porto Rico Irrigation Service, the Bassano Development, for the Canadian Pacific Railway, in Alberta, Canada, the Ozark Dam on the White River, in Missouri, and the La Prele and Shoshone Dams, in Wyoming.

Mr. Coburn made studies of nearly every important river in the United States. At different times he acted as Consulting Engineer to H. L. Doherty and Company, H. M. Byllesby and Company, and other prominent banking houses.

He was a man of unusually attractive personality and had a host of friends, all of whom knew him and loved him as "Pa" Coburn. He was a man to whom other men—especially young men—instinctively went for advice and help, and they always got what they needed. His habit was to be perfectly frank at all times on every subject, but his sympathy and generosity were never failing.

* Memoir prepared by Spencer W. Stewart, Assoc. M. Am. Soc. C. E.

In all his relations, professional and personal, Mr. Coburn's predominant characteristic was absolute sincerity. There was not a square inch of veneer on him, and there was never a doubt of where he stood on any subject. His professional opinions were bluntly stated, and were not subject to review for any consideration, commercial or otherwise.

Mr. Coburn was never married. He is survived by three sisters, Mrs. A. W. Sampson of Rochester, N. Y., Mrs. Charles A. Byram of Patten, Me., and a third who lives in California.

He was a member of the American Society of Mechanical Engineers, and of the Engineers and Technology Clubs of New York City, and the Engineers Club of Boston. He was a life member of Aberdour Lodge, A. F. and A. M., and Aleppo Temple, both of Boston.

Mr. Coburn was elected a Member of the American Society of Civil Engineers on March 5th, 1912.

WILLIAM WATSON COE, M. Am. Soc. C. E.*

DIED MARCH 31ST, 1920.

William Watson Coe was born in Bethlehem, Conn., on November 19th, 1846. He began his studies preparatory to entering the Engineering Profession when he was about fifteen years old. He first made Charles Town, W. Va., his home in 1872, having gone there in connection with the first projection of the Shenandoah Valley Railroad.

Mr. Coe began his professional career in 1866-67, when he held the position of Rodman to Topographer on the surveys for the Dutchess and Columbia Railroad. From 1868 to 1871 he was, first, Rodman and, later, Resident Engineer on the construction of the Dutchess and Columbia Railroad, and in that period was also Assistant and Resident Engineer on surveys and construction of the Connecticut Western Railroad. During the two years from 1872 to 1874 he was Engineer in Charge of surveys for the Plymouth, Kankakee and Pacific Railroad and the New York and Long Branch Railroad; Engineering Assistant to the contractor for the Mobile and Northwestern Railroad; Principal Assistant Engineer on the survey for a branch line of the Atlantic and Great Western Railroad, and Engineer in Charge of the survey of the Carrolton Branch of the New York, Lake Erie and Western Railroad. From 1875 to 1877, he was Superintendent of a division of the Centennial Exhibition at Philadelphia, Pa.

During 1877-78 Mr. Coe served as Principal Assistant Engineer to the general contractor who built the Pittsburgh and Lake Erie Railroad. In 1879 he was made Principal Assistant Engineer of the Shenandoah Valley Railroad, and during the same year became its Chief Engineer, which position he held until 1885. He was appointed Chief Engineer of the Norfolk and Western Railroad in 1881, a position which he continued to hold with that company until 1897. During this period the Norfolk and Western Railroad was increased in mileage by the construction of about 500 miles of new line through the mountainous country in Virginia and West Virginia, reaching the iron and zinc ore territory of Virginia and the Pocahontas coal lands of Virginia and West Virginia. From May, 1897, until February, 1904, Mr. Coe was Engineer of Branch Line Construction of the Norfolk and Western Railway Company.

On February 1st, 1904, he became General Manager of the Pocahontas Coal and Coke Company, having charge of all its coal lands, including about 300 000 acres, with offices at Roanoke, Va. Thereafter, he was continuously engaged in the extension of these holdings, the perfection of titles, and the development of the coal lands.

* Memoir prepared by Charles S. Churchill, M. Am. Soc. C. E.

During his professional life since 1880 Mr. Coe was especially interested in the upbuilding of the southwestern district of the State of Virginia, and considerable areas of coal lands in West Virginia. He was always interested not only in the improvement of the country, but in the welfare of the people in those parts of Virginia and West Virginia, where a large increase in population has taken place since 1881. He was actively engaged in business until a short time before his death, which occurred at his home in Charles Town, W. Va., on March 31st, 1920, as the result of a stroke of paralysis suffered about two months previously. He was buried in Edge Hill Cemetery, Charles Town, on Friday, April 2d, 1920.

Mr. Coe was married in 1880 to Miss Sallie R. Travers, the daughter of the late William H. Travers, of Charles Town, W. Va., who, with a son and a daughter, survives him.

He was a talented engineer and a very capable business man, and was highly regarded by every one with whom he had dealings.

Mr. Coe was elected a Member of the American Society of Civil Engineers on April 3d, 1889.

CLARENCE COLEMAN, M. Am. Soc. C. E.*

DIED OCTOBER 23D, 1918.

Clarence Coleman, son of the late Clayton Glanville Coleman and Sarah Macon Jerdone Coleman, was born at Jerdone Castle, Louisa County, Va., on April 4th, 1849. His father was General of the United States Militia, before the Civil War.

Mr. Coleman was tutored at his home, as a boy, attended several private schools in his State, and then studied at the Virginia Military Institute and at Washington and Lee University, Lexington, Va.

When he was a very young man he was employed for several years (1870-72) by the Houston and Texas Central Railway in various capacities, from Rodman to Resident Engineer, on the location and construction of 200 miles of that road. Then, for 2½ years, he was engaged in private practice at Dallas, Tex. In 1875 he was Engineer of a geological survey for the Buffalo, Bayou and Colorado Railroad, and during the following five years was engaged in private engineering practice at Richmond and Kenmuir, Va. He served in 1881 as Assistant Treasurer of the Richmond Street Railway Company, and in the latter part of that year and in 1882 as Instrumentman on the Chesapeake and Ohio Railway.

He was Engineer of the Arminius Copper Mines Company, and the Sulphur Mines Company, both of Virginia, from 1883 to 1885, during which time he made surveys and installed a complete mining plant for the first named company, and built two branch railways from the mines to the main line of the Chesapeake and Ohio Railway. From May, 1888, until the end of that year, he was in charge of the construction of the terminal docks and transfer bridges of the Housatonic Railroad Company at Wilson's Point, Conn., and, in the same year, made an inspection and report on the company's bridges in Massachusetts.

For a little more than a year, in 1889-90, Mr. Coleman was Chief Engineer of the Santa Fé Southern Railway, and located 48 miles of mountain road over the Ortiz Range.

After a year's private practice as Consulting Engineer at Roanoke, Va., in 1890-91, he entered that city's service, first for four months as Building Inspector, then, for two years (February, 1892, to March, 1894), he was Engineer Member of the Board of Public Works, during which time a sewerage system was planned and built, and pavements and bridges were constructed. While in Roanoke he was a member of the Association of Engineers of Virginia, and in 1892 was elected President of that Association. Two of his contributions to the Association bore the following titles: "The Engineer as a Factor in the Development of Virginia" and "What our Bad Roads Cost Us."

* Memoir prepared by John H. Darling, M. Am. Soc. C. E.

The foregoing record gives an outline of Mr. Coleman's work during the first period of his professional career as a civil engineer, which it is seen consisted mainly of railroad construction and municipal improvements, with intervals of private practice, all of which work was within the Southern States. His activities were now transferred to the northern part of the country, in the Duluth District, where he was engaged during the remainder of his life on Government harbor improvements, carried on by the War Department, with officers of the Corps of Engineers in charge of the District. He was appointed Assistant Engineer by Maj. Clinton B. Sears on November 27th, 1894, and continued to act in that capacity until November 23d, 1917, when he was made District Engineer. He held the latter position until removed by death.

Mr. Coleman's work in this district consisted mainly of marine concrete construction, in which he specialized. First, at Marquette, Mich., Harbor from 1894 to 1898, inclusive, he commenced the construction of a concrete superstructure to the old timber-and-stone breakwater, in accordance with a general design prepared by Maj. Sears, the officer in charge. In this construction the old timber with its rock filling was removed down to about 2 ft. below the water line. Concrete footing blocks, moulded ashore, were laid along the two edges of the timber cribs, and, between and above these, mass concrete was deposited in forms. A gallery traversed the entire length. After a considerable portion had been built, Mr. Coleman was transferred to Duluth, and the work was completed by other engineers.

At Duluth-Superior Harbor, in 1898-1902, he was in charge of the construction of the concrete superstructure for the new combined timber and concrete piers, or jetties, which define the entrance channel from Lake Superior to the north end of Duluth-Superior Harbor. In this case, as at Marquette, there were concrete footing blocks surmounted by monoliths of mass concrete. At the lakeward ends substantial pierheads were built for supporting light stations.

At the south, or Wisconsin, entrance to Duluth-Superior Harbor, also known as Superior Entry, Mr. Coleman, in 1903-09, built some all-concrete revetment piers, which were designed by himself, under the direction of Maj. D. D. Gaillard, Corps of Engineers, U. S. A. The main purpose of these piers was to serve as revetments at the two sides of the entrance channel for retaining the sand of the spit through which the entrance passed. In this construction, which is a departure from the usual types, there are monolithic sections or units, 16 ft. in length, moulded *in situ* around and on foundation piles sawed off a short distance above the level of the bottom, which had been dredged to a depth of 21 ft. below low water. Specially designed removable forms and machinery for handling them were part of the rather expensive plant required for this work.

The construction of these revetment piers was the most original work accomplished by Mr. Coleman, and possibly the most important, also, when its value is considered as an example for similar constructions where conditions are favorable. It has been described in several technical publications.

Among other constructions in the Duluth District, in charge of Mr. Coleman under direction of officers of the Corps of Engineers, may be mentioned the following:

The project of a Board of U. S. Engineers for the further improvement of Superior Entry, approved April 5th, 1907, under which was built, in 1907 to 1912, two converging breakwaters, comprising 4 205 lin. ft. of rubble mould, 900 ft. of stone-filled cribs with concrete superstructure for pierheads, and 896 ft. of concrete shore revetments; the dredging of a stilling basin within the breakwaters with an area of 35 acres (completed in 1915), and the dredging of a harbor basin of 98 acres (completed in 1914); the completion of a 1 500-ft. rubble mound extension to the Marquette Breakwater, in 1914-18.

Mr. Coleman was assigned to the charge of the Duluth District on November 23d, 1917, with the title of District Engineer, following the transfer to another post of Col. D. W. Lockwood, the officer in charge. He held this position until his death in the following year.

Mr. Coleman served in the capacity of Consulting Engineer on a number of private and corporation projects. Owing to his research and experience, he was considered an authority on concrete, and was often called in consultation on this subject.

He was married in 1873 to Miss Bena Randolph Philips, daughter of John Penn Philips, a prominent lawyer of Warrenton, Va., and Elizabeth Lewis Pickett Philips, who was a cousin of Gen. George Pickett, of Gettysburg fame. No children survive. Their union was a happy one, and their home was warm with true southern hospitality. Mr. Coleman was a good liver, and a cheerful disposition beamed from his face. His memory was stocked with a choice assortment of anecdotes and humorous stories, many of them in the negro dialect, which often aptly enlivened his conversation and spiced his public addresses.

He was fond of music, and played the violin in his home. He was a connoisseur of this instrument, and had a hobby for looking up and possessing a collection of old historic violins of rare quality and value which he took pleasure in describing to his friends. In religion, Mr. Coleman was a Roman Catholic, and he and his wife were faithful members of that church.

His final illness extended over a period of four months. Its nature seems not to have been well understood, but hardening of the arteries was said to be the ailment. During the earlier part of his illness he made brief visits to the office, and when unable to leave home he con-

tinued to transact necessary official business at his house. Thus, he "died in harness", at the age of 69.

Mr. Coleman was elected a Member of the American Society of Civil Engineers on November 3d, 1897. At the time of his death he was First Vice-President of the Duluth Association of Members of the American Society of Civil Engineers.

WILLIAM WILLIAMS CREHORE, M. Am. Soc. C. E.*

DIED SEPTEMBER 13TH, 1918.

William Williams Crehore, the son of John T. Crehore and Lucy Williams Crehore, of Cleveland, Ohio, was born in that city on February 3d, 1864.

Those who knew Mr. Crehore intimately held him in high esteem for his great ability in his profession and for the loftiness of his ideals and purity of character. In this he resembled his father, a distinguished civil engineer and a mathematician of high rank, who died on October 7th, 1884.

Mr. Crehore was graduated from Yale University in 1886, having been obliged to lose a year owing to a railroad accident resulting in a broken leg. After 1886, he tutored for a year; he was then given the Larned Scholarship, and returned to Sheffield (Yale) Scientific School for the post-graduate course in Engineering. He took his degree of Civil Engineer in 1888. During the same year he was married to Anna, the eldest daughter of the late Frank W. Ballard, of New York City. He was then for two years Principal of the Hemenway High School, in Norfolk, Va.

In 1889 Mr. Crehore acted as Bridge Inspector for the Pennsylvania Railroad, at Lewistown, Pa., where the Johnstown Flood had made necessary the rebuilding of several bridges. In 1890 he entered the service of Cofrode and Saylor, Bridge Builders, in Pottstown, Pa.; later, he worked for the Baltimore and Ohio Railroad and for the Wallis Iron Works, of Jersey City, N. J., before he opened an office in 1892 in New York City as an independent Civil Engineer.

At just about that time there came into vogue the system of steel frame construction for tall buildings, and Mr. Crehore was one of the first to develop structural designing in that branch of engineering. His resourcefulness and skill as an engineer was shown by the fact that he was called on as a consultant for many large buildings in New York City, in the West, in Cuba, and in the South. The cantilever foundations of the tall American Tract Society Building in New York City were originated and designed by him especially to suit that difficult foundation case. He also developed special designs for foundations under buildings to prevent the menace from quicksand to surrounding structures.

Mr. Crehore, as a public spirited citizen, took a great interest in the advocacy and discussions of the subways in New York City, acting as an adviser to the Merchants Association of New York and also in the public controversies which arose as to the economy and safety of some of the bridges across the East River. He also gave his time and

* Memoir prepared by Gustav Lindenthal, M. Am. Soc. C. E.

services generously to public welfare in his home town, Metuchen, N. J., where he lived before going to California. He was a most intense and enthusiastic worker in anything to which he put his hand, and those who dealt with him had complete confidence in his judgment and thoroughness.

He had a profound love of truth and a loathing for all false assumption. He hated sham, humbug, and charlatanism with all the energy of his soul, and these traits made for him a host of staunch friends.

Mr. Crehore's many-sided nature is shown by the fact that he was an assiduous student of economics and had written a scholarly work, "Protection's Brood", dealing with the problems and economic evils arising from the protective tariff, and particularly pointing out the corruption bred by the excesses of that system. The book gives evidence, not only of painstaking research and analysis, but also, in its easy fluent style, of high literary ability. His discussion is a model of fairness, clarity, and comprehensiveness, and his emphatic language and unsparing criticism of certain evil conditions that had resulted from ruthless competition contributed not a little to their correction, and earned for him a high reputation among thinkers and students of economics. Mr. Crehore properly considered National economics a legitimate field for the study and activity of the Engineer, whose professional interest should be centered through them in the development of public utilities and great public works.

During his college days, Mr. Crehore had obtained a fine musical education from New York's best teachers, and he was passionately fond of classical music. With his fine baritone voice he was a valuable member of the University Glee Club of New York from its organization, in the early Nineties, and sang regularly with it until he left—a sick man—for the West in 1915.

Mr. Crehore was very devoted to his family. He left a widow and nine children—four sons and five daughters. Three of his sons disdistinguished themselves during the World War, in France, in the military service of the United States. One son, Sergeant Austen B. Crehore, volunteered in the French Lafayette Flying Corps, and received for his valor the Croix de Guerre with palm.

Mr. Crehore's long illness prepared his friends for his death, and this he faced with cheerful resignation. With members of his family around him, he passed through days of happiness, in spite of his fatal affliction. He died at Los Angeles, Cal., on September 13th, 1918.

His many lovable qualities, his rectitude, and his fine character will not be forgotten by those who had the privilege of knowing him intimately, and they, with his family, cherish his memory as that of a noble and manly nature.

Mr. Crehore was elected an Associate Member of the American Society of Civil Engineers on April 4th, 1894, and a Member on September 3d, 1902.

JOSEPH BAKER DAVIS, M. Am. Soc. C. E.*

DIED MARCH 9TH, 1920.

Joseph Baker Davis, the son of Ebenezer Hathaway and Mehitabel (Gifford) Davis, was born at Westport, Mass., on July 31st, 1845. His mother died when he was an infant, and he was reared by a cousin of his grandfather, a maiden lady, Miss Anstrus Baker.

Although his ancestors had been seamen, Mr. Davis' guardian directed his attention to other things. He attended the Grammar and High Schools, of New Bedford, Mass., and when he made inquiries regarding a higher education, he learned that one without means might obtain such an education at the University of Michigan. He went to Detroit, Mich., during the summer of 1864 and found employment on a farm not far from the present town of Dearborn, where he assisted in the clearing of a considerable area of land. On October 1st, 1864, he presented himself for admission to the University of Michigan at Ann Arbor. As he had been able to accumulate only enough money to pay the University fees, he realized that he would be obliged to earn his living expenses and buy his books from funds gained by labor as he pursued his studies. Mr. Davis had become accustomed to hardship during his boyhood, and the difficulties which faced him when he entered the University were met with the sturdy confidence which won for him many a victory in the years that followed.

During the summer of 1867, he was employed by the United States Lake Survey in the measurement of the Portage Entry base line, harbor work, Chocolet River reconnaissance, determination of shore line, inland surveys, and soundings, all on Lake Superior. He was graduated in 1868, his class representing a remarkable group of men, which furnished leaders in engineering, in astronomy, and in the teaching profession. In June, 1868, Mr. Davis entered the office of the City Engineer of Detroit, Mich. In order to be out of doors, he soon resigned and accepted employment with a paving contractor, after which he was engaged, for a short time, in the office of an architect. Since office work did not agree with him, he accepted a position with the Missouri River, Fort Scott, and Gulf Railway, in the fall of 1868, and spent eight or nine months in Kansas under the immediate supervision of the celebrated engineer, the late Octave Chanute, Past-President, Am. Soc. C. E. He returned to Michigan in June, 1869, and during the summer and autumn was occupied in running a railway line from Owosso to Big Rapids, a distance of ninety miles. When this work was completed, he returned to Massachusetts for a visit with his relatives after an absence of six years.

In February, 1870, Mr. Davis returned to Ann Arbor, Mich., where

* Memoir prepared by Clarence T. Johnston, M. Am. Soc. C. E.

he was engaged as Instructor in the Department of Civil Engineering, University of Michigan, until the end of the academic year. He then accepted employment with the Toledo and Ann Arbor Railway and, during the summer, made the location surveys for this line. Later, he accepted the position of Assistant Engineer on the Northern Extension of the Jackson, Lansing, and Saginaw Railway, and made the exploratory surveys from Bay City to Gaylord and the location surveys to Grayling.

In the fall of 1871, he went to Swarthmore, Pa., where he organized a Department of Civil Engineering in Swarthmore College, an institution of the Society of Friends. In June, 1872, he was appointed Assistant Professor of Civil Engineering at the University of Michigan, and began his thirty-eight years of active service as a teacher at his Alma Mater. The late Professor De Volson Wood, the Father of Engineering at the University of Michigan, and Professor Davis' former teacher, had tried to re-organize and improve his Department at Michigan. Failing to obtain the help he had many times requested, he resigned in the spring of 1872, and no argument would induce him to reconsider his decision. His recommendations, however, were almost immediately accepted and the late Professor C. E. Greene, M. Am. Soc. C. E., Professor Davis, and Mr. C. S. Denison were at once appointed to carry them into effect.

Professor Davis bore the title of Assistant Professor of Civil Engineering from 1872 until 1891, when he was made Professor of Geodesy and Surveying. For many years he assisted in the teaching of Descriptive Geometry and Drawing, and he gave one or two courses in Municipal Engineering. Although his practical experience had been largely in railway work, he never encouraged students to specialize therein, holding that engineers had been mistreated by the railway companies.

One of the urgent recommendations of Professor Wood related to the establishment of a camp where students might engage in surveying under field conditions. Professor Davis organized and conducted the first camp of this kind at Whitmore Lake, ten miles north of Ann Arbor, in 1874. It was a course of four weeks. The camp moved from place to place each year, depending on arrangments for the land, transportation, and housing facilities that Professor Davis was able to make. In 1882, the camp was transferred to Old Mission and, from that time to the present, it has always been located in the northern part of the Southern Peninsula of Michigan. It remained a four weeks' course until 1902, when it was extended to six weeks and then to eight weeks in 1909. In 1908, the University acquired 1 500 acres of land on Douglas Lake, near Cheboygan, Mich., thus giving the camp a permanent home. The University of Michigan was the pioneer by many years in the establishment of a surveying camp. The camp now

bears the name of its founder, and it is still conducted in conformity with the rules and regulations which he had developed after long years of experience.

Professor Davis became City Engineer of Ann Arbor soon after his appointment as Assistant Professor of Civil Engineering at the University. This position came to him unsolicited and while he was absent from Ann Arbor. Regardless of political changes, he was retained in office for sixteen years, and when he resigned, his assistant was appointed to succeed him. While City Engineer of Ann Arbor he performed a splendid service. He was unable to convince the City Fathers that the notes of property and general city surveys were matters of great importance and that they should be safely kept in a well-equipped office. He retained all such notes, however, and they are now available and highly appreciated.

He acted as Consulting Engineer for the Flint and Pere Marquette Railway Company and the Toledo and Ann Arbor Railway Company for many years, and he designed and constructed many highway and railway bridges. He also installed water-works systems for several Michigan towns, among which were Greenville and Owosso. His great work, however, was performed in the field of Surveying and particularly Boundary Surveying. He always maintained that "corners were never lost, they were simply not found."

From 1899 to 1902, he had charge of the St. Clair Flats Survey for the State of Michigan. This was a difficult task, and it was completed in accordance with the principles he always advocated and practiced.

Soon after the death of Professor Greene, Dean of the College of Engineering, in 1903, Professor Davis was made Associate Dean. His new duties brought him into contact with every student of the College, and many engineers who attended the University of Michigan during the first decade of the present century will remember the gruff but kindly voice of this fatherly adviser. His new duties were more trying to Professor Davis than his friends generally realized, for his natural sympathy for students who suffer privations made it impossible for him to forget their hardships and trials, day or night, and the indoor work added to his difficulties.

In 1910, he resigned his position at the University and took advantage of the Carnegie pension. In order that he might be near his friends, and yet away from responsibilities, he bought a home in Dexter, Mich., nine miles of Ann Arbor. Some time prior to his resignation, he had purchased a summer cottage on Portage Lake, nine miles north of Dexter, but he and Mrs. Davis generally spent their winters in the South, visiting their son at Birmingham, Ala., and then going to the Gulf Coast. They finally secured a winter home at Bradentown, Fla., where he died on March 9th, 1920.

In July, 1872, he was married to Miss Mary Hubbard Baldwin, of

Ann Arbor, Mich. Three children were born to them, one of whom, Charles Baker Davis, of Birmingham, Ala., with Mrs. Davis, survives him.

Professor Davis never sought favor for himself, but was always active in behalf of others. In 1883, he was elected President of the Michigan Engineering Society and was twice re-elected to the same office. He was also for two years Vice-President of the "Council of Engineering Societies on National Public Works."

He had written little for publication, but he left a great volume of notes relating to the theory and practice of Surveying and to other engineering subjects. He always underrated his ability as a writer, but he was an able reviewer and critic of texts relating to the subjects that he taught.

It is a great satisfaction to all who were so fortunate as to have known Professor Davis to understand that he was happy and contented during his later years. He always said that he desired to quit work some time in order that he might play and do many things that were denied him as a boy. His home was always a welcome place for all his friends and acquaintances. He was always ready to perform some kindly service and no guest ever departed without a higher regard for him and for mankind generally.

Professor Davis was the personification of character and stability. He loved sincerity, truth, and accuracy. His former students will always remember him as an advocate and example of those qualities of the mind and heart which guarantee success in the practice of Surveying and which, possibly in a higher sense, insure happiness and contentment in every walk of life. The principles which he advocated and practiced have been accepted by many who have sat in his classrooms and by many others who are now charged with the duties he so ably performed for the University of Michigan. Those who survive him can perform no higher service for the University of Michigan than that which will result in the perpetuation of the ideals and principles which he exemplified.

Professor Davis was elected a Junior of the American Society of Civil Engineers on April 1st, 1874, and a Member on October 5th, 1898.

JOHN STERLING DEANS, M. Am. Soc. C. E.*

DIED DECEMBER 16TH, 1918.

John Sterling Deans was born at Chester, Pa., on June 25th, 1858, and received his early education in the schools of that place. His father, Charles Woodbury Deans, was engaged in educational work and did much in the early organization of the common school system of the State and in providing school facilities for the orphans of Civil War soldiers. His mother, who survives him, was Priscilla Lyons Williams, of Chester.

The death of his father left Mr. Deans, the eldest of four children, with serious responsibilities early in life, and at the age of 20 years he left the Philadelphia Polytechnic College, before graduation, and took up his life work in his chosen profession.

His first employment was with Clark, Reeves and Company, of Phœnixville, Pa., in January, 1879, at which time this firm was building a large part of the elevated railroad structure in New York City; and, during his subsequent connection with the firm, which afterward became the Phœnix Bridge Company, he saw the development of the modern long-span railroad bridge from the early iron structures that were built during the era of railroad construction which followed the Civil War.

Many noted structures, such as the old Kinzua Viaduct, and the Albany and Greenbush Bridge over the Hudson River, with which Mr. Deans was connected at the outset of his career, were built by the above firm. These were followed by the Red Rock Cantilever Bridge, the Pecos Viaduct, the Rock Island Bridge, the Huntington Bridge, and many others, with which he was connected in various engineering capacities.

Mr. Deans assisted in the design of the Chesapeake and Ohio Railroad Bridge, known as the Huntington Bridge, over the Ohio River at Cincinnati, Ohio, had entire charge of the work in the drawing room, and afterward became Resident Engineer during the erection of the structure, in 1888-89.

This bridge, at the time it was built, contained the longest simple truss span in existence, was double-tracked throughout, with double roadways and sidewalks, and marked an epoch in railroad bridge construction. After the successful completion of this work by the Phœnix Bridge Company, Mr. Deans became Engineer of Erection for the Company, then, in 1891, Principal Assistant Engineer, and, a year later, Chief Engineer. During the time he held this office, a number of large and important bridge structures were built under his direc-

* Memoir prepared by Howard C. Baird and Walter R. Okeson, Members, Am. Soc. C. E.

tion, many of them being designed by the Company and most of them erected by its Erection Department, in which Mr. Deans took great pride and which he brought to a high state of development. The Rock Island Bridge at Davenport, Iowa, the Big Four Bridge at Louisville, Ky., the Cambridge Bridge at Boston, Mass., the steelwork for the Manhattan Bridge over the East River, New York City, and the bridges at Sioux City and Omaha, Nebr., with their long draw-spans, were among the large number of structures erected under his supervision.

He became Vice-President of the Phœnix Bridge Company in 1915, and, in 1917, Vice-President and Consulting Engineer.

As a young man, Mr. Deans' work was marked by extreme care and conscientiousness. His optimistic spirit endeared him to all his associates, and in fact to every one who came in contact with him, and inspired many a beginner to look on the bright side of life and take courage from his example. His fine character and lovable personality had their effect in a marked degree on the young men who came under his authority while he was in command, and all who ever served him treasured the memory of his friendship with an affection equal to their great admiration for his character.

About two years before his death, his health began to fail, and, except for several short periods of apparent improvement, the decline was slow, but sure. In spite of his declining health, however, Mr. Deans was as energetic as ever, and gave unfailing attention to the many duties devolving upon him, undertaking in December, 1917, the work of Chairman of the Steel Fabricators of the United States, an association devoted to aiding the United States Government and the industry in war work. His health continued to decline until the end came, on December 16th, 1918. His youthful spirit and courage remained with him to the last, and the patience which he exhibited during his tedious illness and suffering emphasized the perfect unselfishness that marked his life.

In 1890, Mr. Deans was married to Miss Clara V. Barr, of Cincinnati, Ohio, who survives him. He is also survived by his mother, two sisters, and four children, his two sons having served in the United States Army during the World War.

Mr. Deans was elected a Member of the American Society of Civil Engineers on May 4th, 1887.

FRANK LOUIS FULLER, M. Am. Soc. C. E.*

DIED JANUARY 30TH, 1920.

Frank Louis Fuller, the son of Hezekiah and Emmeline (Jackson) Fuller, was born in Grantville (now Wellesley), Mass., on July 11th, 1848.

As a boy he attended Mr. Allen's School in West Newton, Mass., and was graduated from the Massachusetts Institute of Technology in the Class of 1871.

Immediately after his graduation, Mr. Fuller was employed, under Professor J. B. Henck, on the work of the Massachusetts State Harbor and Land Commissioners on the "Back Bay" District, Boston, Mass., until 1881, when Professor Henck resigned, and he was placed in full charge.

In addition to this work, in 1875, Mr. Fuller opened an office in Boston, Mass., and engaged in private practice as a Consulting and Constructing Engineer. As such he made examinations of the Taunton River Valley for the Massachusetts Board of Health, surveys and plans for the proposed Nantasket Beach Railroad, as well as many other general surveys of roads, rivers, etc.

Mr. Fuller early turned his attention to the design and construction of water-works, and made that branch of his profession a specialty. In that connection, he made plans for and constructed water-works plants for various cities and towns in Massachusetts and the adjoining States, among which were Winchester, Wellesley, Ware, Marblehead, Natick, Webster, West Brookfield, Foxboro, Warren, Uxbridge, Mass., Pawtucket, R. I., Brattleboro and Northfield, Vt., etc. He was among the first to be interested in concrete covered reservoirs and had charge of the construction of such reservoirs in Wellesley and Natick, Mass.

He was also engaged on the construction of the sewerage systems of Swampscott, Marblehead, North Andover, Franklin, and other places in Massachusetts.

Mr. Fuller was married on November 23d, 1882, to Julia Latham Morrill, who survives him. Except for a four years' residence (1882 to 1886) in Boston, he had always made his home in Wellesley, Mass., where he died suddenly on January 30th, 1920, from heart trouble from which he had suffered for several years. On his death the Annual Town Meeting of Wellesley adopted a set of resolutions as an expression of appreciation of his services as a citizen and as a member and Chairman of the Board of Water Commissioners from its establishment. Similar resolutions were also adopted by the Congregational Church, the Congregational Club, and by his associates on the Water and Electric Board of Wellesley.

* Memoir compiled from information on file at the Headquarters of the Society.

He was a man of large outside interests and was always interested and active in various good causes. With Mrs. Fuller he had traveled quite extensively, having made two trips to the Panama Canal, the second of which was a visit to California and included a trip through the completed Canal. He also visited Porto Rico and, in 1913, greatly enjoyed a cruise to the Mediterranean and the Orient, including the Holy Land and the Nile.

Mr. Fuller was a member of the Congregational Church of Wellesley, Mass., which he had served as a Deacon for many years. He was a member of the American Water Works Association, the New England Water Works Association, the Boston Society of Civil Engineers, the Wellesley Club, and the Wellesley Congregational Club.

Mr. Fuller was elected a Junior of the American Society of Civil Engineers on April 4th, 1883, and a Member on April 4th, 1888.

CHARLES EMERSON GREGORY, M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1920.

Charles Emerson Gregory was born in Stissing Village, Stanford, N. Y., on February 20th, 1871. He was the son of Edward Benjamin and Julia Wilber (Mosher) Gregory. His paternal ancestor, Johnathan Gregory, came from England in Colonial times and was one of the early settlers of the New Haven Colony. Members of the family served in the Continental Army and one of them fought in the Battle of White Plains. The Mosher family came to America from England soon after the Revolution and settled in New York State.

Mr. Gregory received his early education in the public schools of Stanford and completed his college preparation at the Poughkeepsie Military Institute. He was graduated from Union College, Schenectady, N. Y., in 1894, with the degree of Civil Engineer, and the Engineering Prize.

His first engineering work after leaving college was the construction of a system of water-works at Coxsackie, a drainage canal at Middleport, and water-works at Red Hook, N. Y., all of which was accomplished during 1894 to 1896. In March, 1896, Mr. Gregory entered the employ of the City of New York, in the service of which he was a useful member for many years. At first he was an Engineer Inspector in the Department of Public Works, inspecting the repaving and the laying of water mains in Fifth Avenue; later, he became a Transitman in the office of the Commissioner of the 23d and 24th Wards, now known as the Borough of the Bronx, and, in September, 1898, he was appointed Assistant Engineer in the Sewer Bureau of the Borough of the Bronx where he served about five years in charge of sewer construction and three years in charge of the design of the new sewer system for the easterly portion of the Borough, a drainage area of about 7 000 acres. This included some of the most extensive and difficult sewer work in the City of New York and the design of some of the largest reinforced concrete storm-water drains ever built up to that time. It was then that Mr. Gregory began his studies of rainfall and run-off which continued a matter of special interest to him throughout his whole life. This interest was stimulated by a friendship with the late Emil Kuichling, M. Am. Soc. C. E., which began about this time—a friendship so helpful to both in regard to sewer design and run-off problems. From this time, also, Mr. Gregory was never without a little pocket-book in which were entered all sorts of observations, notes, calculations, and diagrams relating to this,

* Memoir prepared by Arthur H. Pratt, M. Am. Soc. C. E.

to him, ever-absorbing subject. It is to be regretted that a busy life prevented the orderly assembly of this material for publication by the Society, as he had always intended, although much of it has appeared in fragmentary form in discussions of various papers.

In the spring of 1906 Mr. Gregory joined the staff of the Board of Water Supply of the City of New York as Designing Engineer. Under Chief Engineer J. Waldo Smith, M. Am. Soc. C. E., he prepared the designs for the Ashokan Reservoir, including the Olive Bridge Dam and the extensive earth dikes, the Kensico Dam and Reservoir, and the Silver Lake Reservoir. Among the various appurtenant works to these main structures were ten concrete bridges designed in collaboration with the Board's Architect, H. Lincoln Rogers, Assoc. M. Am. Soc. C. E., the most notable of which are the beautiful Traver Hollow and Rye Outlet Bridges—the former a spectacular three-hinged, single arch of 200 ft. span, and the latter a 1 000-ft., five-span structure.

In 1912, on the request of Mr. George McAneny, then Borough President of Manhattan, Mr. Gregory was transferred to the Borough President's office as Deputy to Consulting Engineer Ernest P. Goodrich, M. Am. Soc. C. E. Here, Mr. Gregory was closely associated with all the numerous public improvements proposed and in progress at that time. One of the best known was the removal of the long-standing building encroachments in Fifth Avenue, 42d Street, 34th Street, and elsewhere. This work included not only the approval of the plans and inspection of the execution of the work, but also the delicate matter of securing the co-operation of the owners in spending their own money for the improvement. The Borough President during that administration was a powerful factor in the rapid transit problem and his Consulting Engineer advised on the development of the plans for the new subway systems for the Greater City. Among the many strictly Borough matters taken up at this time were the establishment of new policies for the Bureau of Buildings and the new repaving programme for the streets of the Borough. Plans for remodeling High Bridge over the Harlem River to conform to the War Department requirements were prepared by Mr. Gregory, and the work is now being pushed along lines similar to his original ideas. During the Administration of the late Mayor Mitchel, the Consulting Engineers of the five Boroughs and the Chief Engineer of the Board of Estimate and Apportionment were constituted a Board for the consideration of many of the general engineering problems of the city, and Mr. Gregory served as Secretary to this Board, in addition to his other duties.

In 1913, the City Administration appeared to be ready to take up actively the sewerage problems which had been pressing for a long time. The Metropolitan Sewerage Commission had made its report

recommending the Ocean Island project, much publicity had been given to the subject, and several members of the Board of Estimate and Apportionment seemed particularly interested in the sewerage question. The whole matter was referred to the Board of Consulting Engineers, and after considerable discussion and some controversy finally resulted in the Metropolitan Sewerage Commission making its final report and going out of office, turning over its data and studies to a new establishment called the Bureau of Sewer Plan of the Board of Estimate and Apportionment. This Bureau was under the direction of the Board of Consulting Engineers, Nelson P. Lewis, M. Am. Soc. C. E., Chairman. Mr. Gregory was appointed Secretary and Executive Officer, a staff was organized and a scheme of sewage disposal for the entire city prepared.

In 1914, Mr. Gregory was appointed Chief Engineer of Sewers of the Borough of Manhattan. During the next four years he re-organized the Sewer Bureau on a modern basis, installed businesslike and efficient methods of sewer maintenance, introduced the motor truck mounted pump for cleaning out receiving basins, began the division of Manhattan's sewers into drainage districts leading to a harmonious scheme of sewage treatment, continued the studies of harbor pollution begun by the Metropolitan Sewerage Commission, made many specific recommendations for the mitigation of long-standing nuisances, and had drainage plans prepared for many areas of the Borough where the drainage situation had been particularly bad for years. During this time, the remarkable long-time, Central Park, rainfall intensity records were compiled and studied. Although these records were published by the Rainfall and Run-off Committee of the Municipal Engineers of the City of New York, the data were prepared for publication under Mr. Gregory's direction, as was also the greater part of the Committee's report. The rational method for the design of storm-water sewers was instituted in Manhattan by him and a new run-off curve was adopted, based on the Central Park rainfall curves and sewer gaugings made under his direction, all modified by his firmly established judgment in these matters. A new method of computing the dry-weather flow was also developed, based on the probable future growth of the Borough, now more susceptible of prediction, on account of the newly adopted Zoning Law, and the water consumption determined on a basis of anticipated floor area.

When it comes to the actual authorization of sewer improvements in New York City, it is very desirable to obtain the interest and cooperation of the property owners, as all such work is paid for by local assessment. In this matter, Mr. Gregory was pre-eminently successful, due to his tact and personality, so that by enthusiastic, individual effort on his part he was able to bring many of these improvements

to a conclusion or well beyond the first stages of authorization. Among the sewers finally authorized, the most notable are the West 46th Street tunnel, the East 41st Street storm relief sewer, and the Canal Street District separate system. The use of sewers for snow removal was instituted in New York City while Mr. Gregory was in charge of the Department. He collaborated with the then Commissioner of Street Cleaning, J. T. Fetherston, M. Am. Soc. C. E., in the early snow disposal experiments, and the final success of the sewer disposal method was due very largely to Mr. Gregory's hearty co-operation.

In 1914, Mr. Gregory was sent to Europe by the Borough to observe the sewage disposal plants of France, England, and Germany. On his return, he designed and constructed the first sewage treatment plant to be built in the Borough of Manhattan. It is of the revolving disk, screen type and is located on the Hudson River at the foot of Dyckman Street, in the Inwood District, and is housed in an ornamental building which sets a new precedent in sewerage structures for New York City. The operation results of this plant were published in articles prepared by him for the *Engineering News-Record*. This plant was to have been the first of a series of appropriate sewage treatment units designed to be built at various locations around the borders of Manhattan Island, in accordance with a general harmonious disposal scheme for the Greater City, in the preparation of which Mr. Gregory had been so largely instrumental.

All this work was interrupted by the war, and in April, 1918, Mr. Gregory joined the staff of the Air Nitrates Corporation. He was Assistant Resident Manager at the Muscle Shoals Plant for a time and, later, became Chief of the Civil Engineering Bureau, in which position he was responsible for an enormous amount of building, power-house, machinery, railroad, dock, water supply, drainage, and road construction work in connection with the ammonia nitrate plants at Cincinnati and Toledo, Ohio.

Mr. Gregory retired from this work and returned to New York City in January, 1919, when he was appointed Engineer of Drainage and Sewage Disposal of the Borough of Manhattan and assigned by the late Borough President Dowling to complete his previous studies and finish the long-delayed "comprehensive plan" for the collection and treatment of Manhattan's drainage.

For many reasons it appeared to Mr. Gregory unlikely that the "plan" would be executed for a long time to come, so in June, 1919, he resigned from the City service to accept an appointment as Deputy Engineer with the North Jersey District Water Supply Commission which was beginning the comprehensive development, under State supervision, of the remaining potable waters of Northern New Jersey. In the few months that he was spared on this work, he completed the

designs for the Wanaque Dam and appurtenant works which will make the water of the Wanaque River available as a supply for the City of Newark and neighboring towns. The contract for this work was completed and ready for advertising when Mr. Gregory died at his home in East Orange, N. J., on February 21st, 1920.

The esteem in which he was held in his last engagement is shown by the following resolution passed by the Commission:

"Resolved: The Commissioners of the North Jersey District Water Supply Commission desire to record their appreciation of, and esteem for, its late Deputy Engineer, Charles E. Gregory. They gladly bear witness to his sterling worth, professional standing, and high character. His extensive knowledge of his profession, his knowledge of men and rare tact were of great value in the work of the Commission. His courtly manner and genial disposition made him an esteemed and valuable officer. His work with the Commission, and before such association, was widely recognized as of a consistent and constructive type and remains a lasting memory to him.

"He was of the type of men who leave behind them memories that endure, and we take this opportunity to record in our minutes this testimony of our esteem and affectionate regard for one who was more than an employe, and of his worth as an Engineer and Citizen."

Mr. Gregory was a frequent contributor to the Society's discussions, especially on drainage and water-works subjects. He was also an esteemed writer on these matters for the engineering press. He was a member of the Beta Theta Pi Fraternity, the Masonic Order, the American Water Works Association, the New Jersey Sanitary Association, and was a Past-President of the Municipal Engineers of the City of New York.

For many years, Mr. Gregory had lived at Mt. Kisco, N. Y., where he was an active and honored citizen. He had served as Trustee of the Mt. Kisco Board of Education from 1910 to 1916, having been President of the Board for the last two years, during which time the new main school building was erected and many improvements were made in the school system. He was an Elder of the Mt. Kisco Presbyterian Church for twelve years.

On October 15th, 1896, Mr. Gregory was married to Frances Gurney Sackett, of New York City, and she and their three children—Robert Douglas, Charles Emerson, and Frances Eleanor—survive him.

In personal characteristics, Mr. Gregory was generous, genial, conscientious, and sincere. His generosity was shown by his unfailing respect for and consideration of subordinates; his ever-willingness to listen to their side of the case; his recognition of the most subordinate assistant as a collaborator on every problem. Few engineers can command the respect and admiration, together with the true affection, of their assistants, as did Mr. Gregory. His calm and unruffled per-

sonality were important characteristics; he was always under firm self-control, even under the most exasperating circumstances. The most annoying and troublesome matters were settled fairly, justly, and without malice. He had a winning smile and a kind word for every one, but with these was the firm hand, when firmness was needed.

His gentle persistence, always avoiding needless controversy, but never fearing to keep to the right, enabled him to put through by sheer, personal effort many things where a more openly aggressive personality might have failed. This was especially noticeable in connection with sewer matters in Manhattan where every improvement had to be carried through the many steps of approval by various boards and officials. The citizens of New York City have Mr. Gregory to thank for such improvements as the Dyckman Street screening plant. He had a peculiar ability to accomplish results under the trying and difficult processes of New York's public works system. One of his associates has said:

"Mr. Gregory possessed, in a remarkable degree, the ability to co-operate with others, eliminating all personalities for the end to be accomplished. He was extremely modest in personal demeanor, yet courageous in stating his position on any controversial subject, and he persistently followed fundamental truths and laws in all his activities. He was a man of character, culture, and ability. His capacity for securing results was exceptional."

As an engineer, he was progressive and open-minded and most thorough—every condition was considered, every possible solution investigated. He was always willing to adopt new things if they were good, but never solely on account of their novelty. There was none of the "good enough" spirit about Mr. Gregory—it must be "right". He was an entertaining companion, but disposed to lead the conversation to serious topics. He was open-minded to new ideas in philosophy and government and sincerely interested in all that would improve the social fabric.

Deeply conscientious and serious, with high ideals as to his duties to his family, the community, the State, and to his God, the personality of Charles E. Gregory will forever be a precious memory to every one who had the privilege of being associated with him.

Mr. Gregory was elected a Junior of the American Society of Civil Engineers on October 6th, 1896, an Associate Member on March 6th, 1901, and a Member on June 19th, 1918.

JAMES CURRIE HALLOCK, M. Am. Soc. C. E.*

DIED NOVEMBER 2D, 1918.

James Currie Hallock was born in Treasure City, Nev., on December 8th, 1869. After completing his preliminary studies in his native State, he entered Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in June, 1891, with the degree of Civil Engineer.

Shortly after his graduation, Mr. Hallock entered the service of the Troy Steel and Iron Company of Troy, N. Y., as Assistant to the General Superintendent, remaining until February, 1893, when he entered the employ of the United States Coast Survey Corps which was engaged in deepening the Hudson River for a ship channel, and had charge of the drilling and blasting of a reef at Saugerties, N. Y.

In July, 1893, Mr. Hallock accepted a position with the Government of Ecuador, and until November, 1894, he was Government Engineer and Surveyor-General of the Province of Esmeraldas. In November, 1894, he was employed as Assistant Engineer by the Playa de Oro Mining Company, of Esmeraldas, Ecuador, remaining with that company until May, 1895.

In June, 1895, Mr. Hallock began private practice as Mine Examiner and Expert in San Francisco, Cal., where he remained until August, 1901, when he returned to Ecuador as General Manager of the Southern Exploration and Commercial Company of Ecuador. In October, 1901, he again entered the employ of the Government of Ecuador, as Government Engineer of the Province of Esmeraldas, which position he held until February, 1903, when he was made Director General of Public Works and Chief Engineer for the Government of Ecuador, his duties covering the general supervision of public works, such as the construction of highways, bridges, canals, sewerage and water supply systems, public buildings, etc., as well as the inspection and approval of all railroads and other work being done by contract. Mr. Hallock held this position with great credit to himself and the Government until February, 1906, when a revolution took place, throwing the party by whom he was employed from power. When the Government Palace was surrounded by revolutionists, he succeeded in getting through their lines and saved a number of Government records which he turned over to the American Consul.

Returning to the United States, Mr. Hallock became Engineer in charge of field work with the Passaic River Flood Commission of New Jersey, taking up his duties with this Commission in June, 1906. On the completion of this work, he accepted the position as Assistant Chief Engineer of the New Jersey State Water Supply Commission, and, on

* Memoir prepared by M. R. Sherrerd, M. Am. Soc. C. E.

March 1st, 1909, was made Assistant to the Chief Engineer of the City of Newark, N. J. In February, 1910, the Board of Works created the position of Deputy Chief Engineer, and on February 17th, he was appointed to that office.

In his new position, Mr. Hallock was given direct charge of the meadow reclamation work and the general development of the Port Newark Terminal. The demonstrated availability of the great tract for industrial use is pointed to as a testimonial to his foresight and efficiency. In addition, he was the active head of the City's garbage collection and disposal system, a work in which he also met success.

Possessing a pleasing personality and enjoying widespread popularity, he was active in the civic affairs of Newark, N. J., in addition to his work in an official capacity. He was Chairman of the Executive Committee of the Mayor's Council for National Defense and instrumental in organizing the Home Defense units of the city.

Mr. Hallock severed his connection with the City of Newark in February, 1918, and returned to Ecuador to become Engineer to that Republic. As such his chief task was the suggestion of a system of flood control to curb the annual floods in the River Jubones and to end this serious menace to the chocolate plantations located along that river. Before his death, which occurred in Ecuador on November 2d, 1918, he had submitted a report to the Government for a general scheme to effect such control and was in charge of the inauguration of the work.

He was an active member of the American Society for Municipal Improvements and assisted materially in formulating many of the standard specifications adopted by that Society. He was also a member of the Engineers' Club of New York City and Vice-President of the Traffic Club of Newark, N. J.

Mr. Hallock was elected a Member of the American Society of Civil Engineers on October 3d, 1906.

WILLIAM JOSEPH HARDEE, M. Am. Soc. C. E.*

DIED NOVEMBER 17TH, 1918.

William Joseph Hardee was born on March 27th, 1863, in Brandon, Miss., at which place his parents resided temporarily during a part of the time of the Civil War. He was the first of ten sons born to Thomas Sydenham Hardee, of Savannah, Ga. (himself a Civil Engineer of high reputation), and Harriot Sophia Zacharie, of New Orleans, La. He was a nephew of Gen. William Joseph Hardee of the Confederate Army, and was named for that distinguished man.

In early childhood Mr. Hardee's parents moved from Brandon to New Orleans, and there he received his preparatory schooling. After his professional education, his first work was with the old New Orleans and Pacific Railroad, later known as the Texas Pacific. In 1866 he became connected with the United States Engineering Department, as Assistant Engineer, and for nearly fifteen years was engaged on hydrographic and topographic work on the Mississippi River from Vicksburg to the mouth, including charge of the location, construction, and maintenance of the great levee system in that territory.

In 1899 Mr. Hardee was elected City Engineer of New Orleans, which position he held until his death. During his eighteen years as City Engineer, New Orleans undertook and accomplished more construction work than in any other period of its history, and many of its prominent public works are monuments to his skill and professional ability. It was under his administration and supervision that the great harbor belt-line project was started and completed, the chain of old public school buildings replaced by modern structures, the street paving throughout the city brought up to metropolitan standards, including a municipal repair plant, and many of the public parks and playgrounds created. Notable among these improvements is the sea wall and park at West End, which was designed and constructed under his direction.

Capt. Hardee died after a brief illness resulting from epidemical influenza. His life had been devoted to his profession in which he had attained a place of distinction and National prominence. He was a veteran of the Spanish-American War, having volunteered for service at the outbreak of those hostilities, and was commissioned by President McKinley, Captain of Company F, Third Regiment, U. S. Volunteer Engineers.

Capt. Hardee was a Member of the Louisiana State Society of Engineers, and of the National Association of City Engineers. He was twice President of the latter Association, and at the time of his death was an Honorary Vice-President of the organization. He was elected a Member of the American Society of Civil Engineers on May 6th, 1896, and was the oldest New Orleans member of the Society.

* Memoir prepared by George W. Hardee, Esq.

JAMES HAYWARD HARLOW, M. Am. Soc. C. E.*

DIED OCTOBER 6TH, 1918.

James Hayward Harlow, was born in Westminster, Worcester County, Mass., on September 30th, 1846. He died at his home at Darlington, in the Fifth District of Harford County, Maryland, on October 6th, 1918. Like many another of the old New England stock, he could trace his ancestry to the *Mayflower,* being the tenth in descent from Richard Warren and the ninth from John Alden and Priscilla Mullins.

His father, Noah R. Harlow, was a civil engineer, and was engaged on the location of the Nashua and Lowell Railroad, one of the first roads built in the United States. Later, he was Assistant Engineer in charge of the Fitchburg Division of the Vermont and Massachusetts Railroad, and, in 1854, Chief Engineer of the Terre Haute, Alton and St. Louis Railroad. In 1856 he returned to Massachusetts and for much of the time until his death in 1892 was Engineer and Paymaster of the Proprietors of Locks and Canals, at Lowell, Mass.

His mother was Sarah Ann Hunt, daughter of Asa and Maria F. (Hollis) Hunt. She died before he was a year old, and his father married Sarah Caroline Howe. Under her charge he never lacked a mother's love and care.

As a boy James Hayward Harlow was not strong, and attended school but little, having been taught at home. His mathematics he acquired by private study, and his primary engineering knowledge by helping his father. In 1863, he worked in a small machine shop in Leominster, Mass. While here he gained such skill in the handling of tools that, later in life, when he did a little filing, the machinists remarked that he had done that thing before and knew how to hold a tool.

In 1864 Mr. Harlow's family moved to Lowell, Mass., and he was employed for about a year in the office of the Proprietors of Locks and Canals. In 1865 he commenced his engineering work in the employ of the Franklin Company, at Lewiston, Me., the corporation owning the water power of the Androscoggin River. He remained there two years. In 1867 and 1868 he was at Davenport, Iowa, in the employ of the United States Government, on the improvement of the Mississippi River. In 1869 he was under Gen. Rodman, in the U. S. Arsenal at Rock Island, Ill., and he also spent some time in this vicinity on a railroad survey. In 1871 he returned to Lowell and was Assistant Engineer in charge of laying the pipe lines of the original plant of the Lowell Water-Works. In 1872 he was Assistant Engineer in charge

* Memoir prepared by George R. Harlow, M. Am. Soc. C. E.

of laying the pipe lines for the "new" water-works at Pittsburgh, Pa., and remained there until May, 1874.

Mr. Harlow then opened an office in Boston, with the late Gorham P. Lowe, M. Am. Soc. C. E., as partner, but this partnership was dissolved in 1876. For the next two years he was with the engineering force on the Metropolitan Water Supply, being at South Framingham, Mass., for much of the time. In August, 1878, he was appointed Assistant Engineer under Lieut. Mahan, who had charge of building the Davis Island Dam, in the Ohio River just below Pittsburgh. The pool made by this dam extends to Dam No. 1, on the Monongahela River, and makes a harbor for the coal boats at Pittsburgh. Under Mr. Harlow's supervision, the lock walls were built and the foundations for the dam commenced. The appropriation having been spent, he was asked to resign in January, 1880.

Again he opened an engineering office, this time in Pittsburgh, Pa., where he made his headquarters until he moved to Maryland, in 1902. He helped to organize the New Castle Water Company in 1881, and designed and superintended the building of its water-works at New Castle, Pa., in 1881 and 1882.

From 1881 to 1885 Mr. Harlow was Chief Engineer of the Monongahela Navigation Company, in charge of the construction of the dams and locks on the Monongahela River. Under him the new Dam and Lock No. 7 and the second lock at Dam No. 3 were built. In 1884, he was made President of the Water and Gas Works Construction Company, with offices at Pittsburgh. This Company built the water-works at Elkhart, Ind., in 1884, Eau Claire, Wis., and Mattoon, Ill., in 1885, and Latrobe and Mercer, Pa., in 1886. In this latter year they built a large gas pipe line through some of the principal streets of Pittsburgh. This contract was disastrous, and the Company was dissolved in 1887. In this year he entered into partnership with his two younger brothers, George R. and Edward J. Harlow, under the firm name of James H. Harlow and Company, Engineers and Contractors. They built water-works in Canisteo and Port Henry, N. Y., Tarentum, Etna, Waynesburg, Millvale, and Homestead, Pa., and Annapolis and Bel Air, Md.

In 1888, Mr. Harlow acquired the charter of the Pennsylvania Water Company, the company which now supplies the municipalities adjoining Pittsburgh on the east. Under his management, the capitalization of this company grew from $300 000 to $2 250 000. In 1889, when the works were first built, there were 14 miles of pipe lines, none larger than 16 in. In 1902 there were 102 miles of pipe lines, including about 3 miles of 42 and 36 in. In this latter year he sold his interest in the corporation to the Mellons, of the Mellon National Bank of Pittsburgh.

In 1894 Mr. Harlow's attention was called to the water-power possibilities of the Susquehanna River between Columbia, Pa., and its mouth. From being an engineer he became also financially interested in acquiring properties along its shores. He and his associates sold their holdings in Pennsylvania to the builders of the dam at Holtwood, now operated as the Pennsylvania Water and Power Company. In 1915, this company delivered in Baltimore 343 000 000 kw-hr. of electric power.

After 1904 Mr. Harlow resided at Darlington, Md., and was Chief Engineer of the Susquehanna Power Company, which controls the Susquehanna River from the tail-race of the dam at Holtwood to its mouth. When this part of the river is developed, it will produce twice as much power as that delivered by the Pennsylvania Water and Power Company in 1915. He owned all the capital stock of the Havre de Grace (Md.) Electric Company, and was its Treasurer. He also owned the electric plant at Perryville, Md., which he operated as the Perryville Electric Company. In addition to the enterprise just mentioned, his advice was sought and given in many water-power and other hydraulic cases.

Thus far, reference has been made only to Mr. Harlow's engineering work. While at the Davis Island Dam he was one of the organizers of the Engineers Society of Western Pennsylvania, and was its first Secretary, from 1880 to 1884, and again in 1891. He was a member of the Engineers Society of Pennsylvania, of Harrisburg, and of the Engineers Club of Baltimore. He was one of the founders of the Pennsylvania Water Works Association, had been its President, and, up to 1902, had served on its Executive Committee.

Mr. Harlow was one of the Charter Members in 1890 of the Edgewood Church, of the Pittsburgh Presbytery, and was President of its Board of Trustees until 1902. He was a member and supporter of Harmony Presbyterian Church, near Darlington, a member of Darlington Grange, the Darlington Good Roads League, the Darlington Civic Association, the Darlington Cemetery Company, and the Community Council of Defense, and organized the Darlington Creamery Company. His ability as a farmer is shown by the fact that when he bought his home place, of 32 acres, it would not keep three horses and two cows without having to buy feed; but in 1914 he built a large barn, and the place now sustains three horses and eight cattle. Mr. Harlow had served as Foreman of the Grand Jury of Harford County, and had investigated rumors of misdoings among the officials, to the satisfaction of the Court and County.

These connections show how widely his interests extended. How he was esteemed by his associates is shown by the following memorials presented after his death.

The Pennsylvania Water Works Association said:

"And because of his genial, lovable personality, his presence was always welcome."

The Directors of the Baltimore County Water and Electric Company said:

"The Board of Directors have had taken from their number a sincere and beloved friend and adviser, whose broad experience and ripe judgment was ever sought and relied upon. His hearty cooperation, coupled with his untiring energy and constant alertness for the upbuilding and success of the enterprise in which he had so keen an interest, was always at the command of the Company, and frequently at great personal sacrifice.

"Always willing and anxious to serve in any capacity for the general good and substantial progress of the community of which in the later years of his life he so completely made himself a part, the State of Maryland has lost a man whom it will be difficult to replace."

The Darlington Grange said:

"* * * Brother Harlow put his trust in God, living a quiet, undemonstrative, but effective Christian life. He nurtured hope, being essentially optimistic, both by nature and conviction. He dispensed charity, was kind to those in need, especially seeking to help men to help themselves. And in all this community, and wherever he was known, he was noted for his fidelity; honorable in all business transactions; faithful to every covenant and agreement."

The Home and Foreign Missionary Society of Harmony Church said:

"* * * We came to appreciate his worth as a Christian man, and the value of his counsel and aid in all our church enterprises. And we are deeply sensible of the loss which the Church as well as the community has sustained by his removal."

The Community Council of Defense, of Dublin Precinct, Fifth District of Harford County, said:

"We cherish the memory of his noble character, his fine Christian spirit, his lofty patriotism, and the wisdom of his counsel."

John A. Walls, M. Am. Soc. C. E., Chief Engineer of the Pennsylvania Water and Power Company, in a letter to Mrs. Harlow, said:

"I feel that in Mr. Harlow's death I have lost a great friend and the world a business man and engineer of high integrity and ability. I cannot tell you how much I have appreciated the friendly counsel given from his mature experience. With pleasure I looked forward to his occasional visits here, for he was always willing to help us somewhat younger men in those of our problems that had been his. His ready acceptance of the new, tempered by a just appreciation of the older established principles in engineering work, made his viewpoints particularly interesting and valuable. I count it a privilege to have known him."

He was married on April 11th, 1872, to Thursa Alice Butler, of Pelham, N. H., who, with their four children, survives him. The oldest child is James H. Harlow, Jr., Assoc. M. Am. Soc. C. E., President of the Havre de Grace Electric Company, at Havre de Grace, Md. The second is Florence Harlow, wife of George F. Rowell, M. Am. Soc. C. E., of Philadelphia; the third is Justin E. Harlow, in charge of the electric current at a sugar refinery at Clinton, Iowa; and the fourth is Miss Alice Harlow, who is a graduate of the Nurses Training Course at Johns Hopkins Hospital, Baltimore. He had eleven grandchildren.

Mr. Harlow was elected a Member of the American Society of Civil Engineers on March 4th, 1874.

WILLIAM APPLETON HAVEN, M. Am. Soc. C. E.*

DIED JANUARY 6TH, 1919.

William Appleton Haven, the twelfth child of William Haven, merchant, and Sophia (Henderson) Haven, was born in Portsmouth, N. H., on August 12th, 1831. One of his ancestors, Richard Haven, who came to Lynn, Mass., from England on August 23d, 1643, was a carpenter, and worked in several of the new settlements. It is recorded that he built a house in Boston, and refused as part payment therefor a considerable tract of land in that city. As Boston then consisted of three hills, separated by swamps, the prospective value of the land offered did not appear to be as great as that of the house. He went inland about 15 miles, to what is now Framingham, known then, as now, as the "grassy lands", well watered by springs, and there settled and married.

On August 23d, 1843, William Appleton Haven went with his father to Framingham, to attend the second centennial anniversary of the arrival of Richard Haven in America. There were present more than 1 000 of his descendants, comprising the majority of the foremost inhabitants of that part of Middlesex County in which Framingham is situated. No very rich people were among them. They were farmers, tradesmen, ministers, doctors, lawyers, members of various conventions, and early legislators, and it was noted that, for nearly two centuries, there had never been a year in which some descendant of Richard Haven had not been a student at Harvard College.

At about the same time as Richard Haven arrived in Lynn, Thomas Appleton, who traced his ancestry in England back several centuries, came to Ipswich, Mass., and there built a tide-water mill. Many of his sons were graduated from Harvard, and, in 1750, Nathaniel Appleton, D. D., LL. D., was President of that college. His daughter, Mehitable Appleton, was married to the Rev. Samuel Haven, D. D., in 1753. One of the children of this marriage was William Haven, the father of William Appleton Haven.

The elder Haven, at the age of fifteen, was sent to a West India port as a supercargo, continuing to other foreign ports, until the embargo under President Jefferson put a stop to all foreign commerce. He then organized and conducted the first marine insurance company of Portsmouth. When foreign business began again he became supercargo for W. H. and J. Haven, making trips to Calcutta, Marseilles, Buenos Aires, and Savannah. In 1820 he was appointed cashier of the New Hampshire Bank, continuing until the expiration of the charter in 1852.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

William Appleton Haven attended private schools until he was fifteen, and then went to Boston, where he was apprenticed for two years with the firm of Brown and Hastings, Civil Engineers and Land Surveyors. On May 10th, 1849, he left Boston, and traveled by railroad to Montpelier, Vt., the temporary terminus of the Central Vermont Railroad, thence by stage to Burlington, by steamboat to Plattsburg, and by stage to Ellensburg, where he took a position as Rodman on the Northern Railroad of New York, then being built from Ogdensburg to Rouses Point (Ogdensburg and Lake Champlain Railroad). The next year he became Leveler, and remained on that railroad until its completion, in the fall of 1850. He then returned to Portsmouth to pursue his studies in civil engineering.

Early in May, 1852, with $50 in his pocket, Mr. Haven started for the West, to get engineering work. His money was soon spent, but his pluck increased, and in June, after many disappointments, he at last found himself in Marquette, Mich., without a cent in his pocket, but with an engagement as Transitman on the survey of the Marquette and Green Bay Railroad, the first railroad projected on the shore of Lake Superior. He was employed on this survey until the work stopped, in August. In the following October, he became Assistant Engineer on the Galena and Chicago Union Railroad, and then Resident Engineer and Roadmaster on the Beloit and Madison Railroad, remaining until 1857. Then he took charge of a locating party on the Milwaukee and La Crosse Railroad. These lines are now parts of the Chicago, and North Western Railway.

During 1858 Mr. Haven was Principal Assistant Engineer of the Little Rock and Fort Smith Railroad, in Arkansas, now part of the Missouri Pacific Railway. In 1859 he was a member of the firm of Haven and Ames, Surveyors and Civil Engineers, in private practice in Bloomington, Ill. In the spring of 1860 he was appointed Chief Engineer of the Houston, Trinity and Tyler Railroad, now a part of the International and Great Northern Railroad, but construction was suspended in December, 1860, "until the troubles in South Carolina blow over." Mr. Haven's headquarters and residence were at Harrisburg, a small village about 6 miles south of Houston, Tex., and he, being a legal voter, as well as a Northern man, in January, 1861, was honored by being taken in a carriage, with an armed escort, to the voting place, where he cast one of the 124 ballots "for secession" (the vote in that town was unanimous). The next day he was requested (a "request" to a Northern man in Texas from the Committee of Safety in 1860-61 would nowadays be called an imperative "order") to join the "Harrisburg Guards", in charge of Capt. "Poker" More. Mr. Haven never knew Capt. More's real name, but his cognomen was given on account of his proficiency in his favorite game.

As a private in this company, Mr. Haven drilled three times a week, and so learned "Scott's tactics." At that date there were neither railroads nor telegraphs connecting Texas with the other States, but steamships arrived at Galveston twice a week with mails, and the news was immediately telegraphed to Houston. On steamer days all the male population of Harrisburg gathered around the railroad station where the telegraph operator intercepted and read aloud the news from the "States." Mr. Haven used to join in the cheers for the South and hisses at the Northern mudsills, "who are afraid to fight."

One morning in April, 1861, the operator read off something that raised louder cheers from the crowd, but made Mr. Haven realize that Texas was no place for him: the news from Fort Sumter; the call for 75 000 men; the movement of the regiments of Massachusetts; the action of the Legislature of New York; and that the whole North was responding to the call of President Lincoln. Following this came orders for the Harrisburg Guards to be ready to start for Galveston the next day, and thence "on to Washington." Luck and pluck enabled him to desert ignominiously from the Guards that evening and catch the steamboat for Galveston. The next day at noon he was on an ocean steamer, outside of Galveston bar, bound for New Orleans. Thence he traveled quickly by river and railroad to Portsmouth.

Mr. Haven remained at home with his mother and sisters during 1861 and until August, 1862, and then, with Joseph H. Thacher and George T. Wilde opened a recruiting station and called for volunteers. They set up a tent on the Parade, hoisted a flag, and got a disabled soldier of the 2d New Hampshire Volunteers to teach "Hardee's Tactics" and drill them and the recruits in the evenings in Jefferson Hall. In October of that year, with about 75 men, they went to the State camp in Concord, and soon became known as Company K, 16th New Hampshire Volunteers, Mr. Haven being Second Lieutenant.

Lieut. Haven was for a time on duty with his regiment at Butte-a-la-Rose, La., but when the fort was abondoned and the barracks burned, he was delirious with fever and had to be carried to the steamboat. Still burning with fever on arrival at Algiers, Lieut.-Col. Fuller and a lieutenant of Company A, carried him to New Orleans, where he was attended by Dr. Lurton, a skilful resident physician. Lieut. Haven, too sick to join the regiment when it was ordered home, was sent to New York by steamship. He met his regiment at Concord, and was mustered out of service on August 20th, 1863. His fever was followed by dropsy and then by varicose veins.

During the fall and winter of 1863, Mr. Haven bandaged his leg and limped about Portsmouth, but in the early spring of 1864 he obtained employment as Assistant Engineer on a railroad in Connecticut. Then, in the summer, finding that he could work 14 hours a day, and take long tramps, and thinking he could still render some service

in the army, he resigned his position and returned to Portsmouth. There he recruited a few young men and went with them to Concord, where he received a commission as First Lieutenant of Company D, 18th New Hampshire Volunteers. His regiment went to Virginia, and he remained on duty until mustered out in June, 1865.

After passing the summer in Portsmouth and Gosport, Mr. Haven again became actively engaged in engineering. During 1866-67 he was Locating Engineer on the Oswego and New York Midland Railroad (now New York, Ontario and Western Railroad); in 1868-69 he was Chief Engineer of the Dixon, Peoria and Hannibal Railroad, in Illinois; and in 1870-71, Chief Engineer of the Chicago and Michigan Lake Shore Railroad, a line from Chicago to Holland, Mich.

In 1871 he was appointed Chief Engineer of the Buffalo, New York and Philadelphia Railroad (now part of the Pennsylvania Railroad), locating and constructing the line between Olean, N. Y., and Emporium, Pa., on the Philadelphia and Erie Railroad, remaining on this work until the end of 1872.

During the winter of 1872-73 he was Assistant Engineer of the New York Central and Hudson River Railroad, stationed at Rochester, N. Y., on double-track work. On April 25th, 1873, he was appointed Chief Engineer of the Wisconsin Central Railroad, and stationed at Stevens Point, Wis. The panic of 1873 caused the suspension of all work on this railroad, and in 1874 he returned to Rochester, N. Y., where he owned a home. For several years he was engaged in a manufacturing enterprise, but took up engineering again in 1877, when he was appointed Chief Engineer of the Brattleboro and Whitehall Railroad, a line in Southeastern Vermont, now operated by the Central Vermont Railroad.

In the fall of 1878 Mr. Haven was appointed by the Second Vice-President of the New York, Lake Erie, and Western Railroad (now the Erie Railroad), as Engineer in charge of construction of the Buffalo Division. He commenced his duties in Buffalo, N. Y., in January, 1879, having had charge of double-tracking from Burns to Attica, Buffalo terminals, elevators, docks, coal pockets, East Buffalo improvements, and shops. This work was stopped by order of President Jewett in July, 1882. Early in that month, the Buffalo Creek Railroad decided to make extensive improvements, such as double-tracking, rearranging grades, building new iron bridges in place of the old wooden ones, etc., and Mr. Haven was appointed Engineer in charge of construction. Other work was the building of a coal-loading trestle, for shipping coal by lake, which fronted on the City Ship Canal, and an extensive shore protection to prevent the erosion of the Company's land by the waves of Lake Erie. On the completion of the work, on November 1st, 1884, he resigned.

Then, in partnership with S. T. M. B. Kielland, M. Am. Soc. C. E., he opened an office for the private practice of engineering and land surveying. Much of his work consisted of obtaining data in relation to grants, to railroad corporations, of lands and rights in streets and other public property, as well as the boundaries of certain waterways, most of this work having been done formerly by Mr. Peter Emsley, for many years a most reliable and expert surveyor in Buffalo. It was necessary to read the "Proceedings" of the Common Council of Buffalo, and consult the old maps and other records of the City Clerk and the County Clerk of Erie County. Mr. Kielland looked after the maps and Mr. Haven read the 34 volumes of proceedings. In the meanwhile, they were engaged in surveying and various engineering employments along the Niagara River and the Buffalo docks. The partnership was dissolved in 1885, Mr. Haven remaining in Buffalo a year longer. In March, 1886, he received an appointment from the late Gen. Adna Anderson, M. Am. Soc. C. E., Chief Engineer of the Northern Pacific Railroad, and removed to Montana. In the summer of 1886 he was in charge of location and construction on the Helena and Red Mountain Railroad, and his jurisdiction was subsequently extended over the Helena, Boulder Valley and Butte Railroad, the Marysville Branch, Elkhorn Branch, Drummond and Phillipsburg Branch, and others of the Northern Pacific lines in Montana, all mountain roads in the mining regions, generally requiring skilful location and the solution of many difficult engineering problems.

Mr. Haven had a painful and trying experience in 1887, when on a visit to Buffalo. He was stopping at the Richmond Hotel, when, on the morning of March 17th, it was completely destroyed by fire. He was awakened by the electric bell in his room, on the third floor, and, hearing cries of "fire", took up his clothes and ran. He fell over a body at the foot of the stairs on the second floor, but picked himself up and rushed for the balcony, from which he was rescued by firemen. He was burned severely, especially about the feet, legs, and head, lay helpless for thirteen weeks, and was then taken to Eastern Massachusetts, to the home of his sisters (his father died in 1856, his mother in 1879, and the old home in Portsmouth was gone), thence to Cape Ann, where new cuticle was formed and his health was restored.

As the Northern Pacific Railroad Company, according to its charter, could not build branches until its main line was completed to Tacoma, it organized the Northern Pacific and Montana Railroad Company, of which Mr. Haven, on his return to Montana, was appointed Engineer, on June 21st, 1888. In this capacity he directed the partial location and construction of two important branches, the Butte line of the Northern Pacific Railroad, from Logan to Butte, 70 miles, crossing the main range of the Rocky Mountains and forming a second and alternate line between Logan and Garrison; also the Cœur d'Alêne line,

from near Missoula Point to the mining region of the same name in Idaho. The latter was about 120 miles long, and included a very difficult crossing of the Bitter Root Mountains.

In beginning the Montana surveys, Mr. Haven asked Gen. Anderson, "How steep a grade can we use on these branch lines, and how sharp a curve?" The reply almost staggered him, after being used to the maximum 0.5% grade and 4° curves on the Erie Railroad. Nevertheless, a 4% grade and a 22° curve had to be used on the Boulder-Elkhorn line.

Mr. Haven found it difficult to get locating engineers who could make grade-line surveys, and was getting impatient, when he discovered that E. H. McHenry, M. Am. Soc. C. E., one of his assistants, could do the work. This was the beginning of the career of Mr. McHenry, who succeeded Mr. Haven, and was afterward Chief Engineer of the Northern Pacific Railroad.

On May 13th, 1890, Mr. Haven was appointed Superintending Engineer of the West Gallatin Irrigation Company, in charge of the canals of the company in the valley of the West Gallatin River, in Gallatin County, Mont., which comprised about 150 miles.

Following this engagement, Mr. Haven, during 1893-95, was Chief Engineer of a railroad, promoted by Mr. Richard Harlow, planned to run from Helena, through White's Gulch to White Sulphur Springs, and probably farther east. This line was graded from Helena to near Canyon Ferry, but was never completed.

In writing of railroad building in Montana, before and after 1887, John J. Donovan, M. Am. Soc. C. E., says:

"E. H. Beckler, what a world of reminiscence his name arouses? He in charge of the Montana Central Railway, later the Great Northern, I for the Northern Pacific. Both lines under construction through the same canyons to Butte, crossing in the field, fighting in the Courts, yet with no bad blood between the two corps of engineers. Then, when finally Helena welcomed the advent of the Montana Central, we of the Northern Pacific decorated our office with the rest, hung out a big banner across the front: 'N. P. Pay your money. Take your choice. M. C.', and cheered James J. Hill and Col. Broadwater as they passed. Their hats went up in acknowledgment and they passed on. So we built for our companies and kept faith without loss of friendship."

The Montana Society of Civil Engineers was organized in Helena, on July 5th, 1887, with forty-eight charter members. Col. Joseph T. Dodge was the first President. Mr. Haven was the seventh President, filling first the unexpired term of Col. Walter W. De Lacy, and held the office during 1893 and 1894. From the time that he became President he was active in holding the Society together. During his presidency he adopted the unique system of naming but one member on a committee, claiming that one member would secure better and

quicker results than a combination. Mr. James S. Keerl, who was President in 1895, writes of Mr. Haven:

"He was a most genial companion, a gentleman of the old school, and a most entertaining and instructive talker. He was the life of the Society's meetings, and never missed one where it was at all possible for him to attend. He took a lively interest in all classes of engineering, and delighted in discussing any question of engineering presented. The Society owes him a most grateful appreciation for his sterling loyalty and for his earnest work given freely at all times in its behalf, and during the years of its infancy when it needed it most, and those who knew him best enjoyed a deep sense of gratification through his early enrollment among the Honorary Members of the Society."

It may be opportune to state here that Mr. Sigismund Low, the writer's father, was in Helena in 1871, where he had charge of the location of the Northern Pacific Railroad in Montana, under the late W. Milnor Roberts, Past-President, Am. Soc. C. E., Chief Engineer.

During 1886 and part of 1887, the late William H. Kennedy, M. Am. Soc. C. E., was an Assistant Engineer of the Northern Pacific Railroad under Gen. Anderson, engaged on location work through the Prickly Pear Canyon and Boulder Valley.

Mr. Haven was one of the early members in the first organization of Storer Post No. 1, G. A. R., of Portsmouth, N. H., and in 1889 became a comrade of Wadsworth Post No. 3, Department of Montana. He was Quartermaster of Wadsworth Post from 1892 to 1896, inclusive.

In the spring of 1896, Mr. Haven, finding it necessary for climatic reasons to leave Montana, returned to the lower altitude of Buffalo, N. Y. William F. Merrill, M. Am. Soc. C. E., who in 1870 had been one of his Assistant Engineers, was then Vice-President of the Erie Railroad, and appointed Mr. Haven Inspecting Engineer, to look after the interests of that company in connection with the elimination of grade crossings of various streets in Buffalo. In this work other railroads also joined, the whole being done under the direction of the Grade Crossings Commission. He also acted as Engineering Expert in law cases in which the Erie Railroad Company was involved. He retained his connection with this railroad until his death.

When Mr. Haven returned to Buffalo he became greatly interested in the Engineers' Society of Western New York, and was its President for several years, which included 1901, when the Society held its convention at Niagara Falls.

Owing to the marked attention accorded by him to distinguished visitors from Chile to the Pan American Exposition during that year, he was elected on November 27th, 1901, a Corresponding Member of the Instituto de Ingenieros de Chile, Santiago, Chile.

The following concerning this appointment is taken from the *Buffalo Evening News* of March 8th, 1902:

"President William A. Haven of the Engineers' Society of Western New York and Mr. George A. Ricker of that Society, have been elected members of the 'Institute of Engineers of the Republic of Chile.' Word of this honor to two distinguished Buffalo engineers was received in this city from South America yesterday, and it is understood that this action was taken by the famous Chilean National Engineering Association in recognition of the courtesies shown to visiting engineers of South America by the Engineers' Society of Buffalo during the continuance of the Pan American exposition.

"Mr. Haven, as president of the local association, and Mr. Ricker, as chairman of the Reception Committee, during the Exposition season came in personal contact with several widely known South American engineers and the visitors were much delighted with the arrangements made by the Engineers' Society of Western New York for the entertainment of strangers who were members of the profession.

"They expressed their gratification with a warmth characteristic of their race and it is evident that the memories carried back with them were cordial ones.

"Among the distinguished visitors from Chile was Senor Santiago Sotomayor, who is engaged in a tour of the world in the interests of his government, collecting information for a report on electric railways. Senor Sotomayor is a son of Gen. Sotomayor who was commander-in-chief of the Chilean forces in the recent war with Peru. He was particularly impressed with the electrical progress of Buffalo and Niagara Falls, as well as most appreciative of the courtesies extended to him, and it is believed that the honor bestowed on the Buffalo engineers and the society they represent has come through the suggestion of Senor Sotomayor."

On December 27th, 1870, Mr. Haven was married to Zaidee B. Hartupee, the youngest daughter of Dr. Andrew W. Hartupee, of Chicago, Ill. They had no children.

While in Montana Mr. Haven was a member and one of the Trustees of the First Unitarian Society of Helena.

Many engineers who were assistants of Mr. Haven became prominent later: the late James Edmund Childs, M. Am. Soc. C. E., became President of the New York, Ontario and Western Railroad; the late Robert Lewis Harris, M. Am. Soc. C. E., became Vice-President of the Erie Railroad and later President of the Northern Pacific and Chicago, Burlington and Quincy Railroads; William Fessenden Merrill, M. Am. Soc. C. E., became Vice-President of the Erie Railroad; Edwin Harrison McHenry, M. Am. Soc. C. E., became Chief Engineer of the Northern Pacific Railroad; the late Charles Latimer, M. Am. Soc. C. E., became Chief Engineer of the Atlantic and Great Western Railroad (New York, Pennsylvania and Ohio), and the late William Edwin Hoyt, M. Am. Soc. C. E., became Chief Engineer of the Buffalo, Rochester and Pittsburgh Railway, and there were many others.

Mr. Haven's natural intelligence, and his long experience and association with prominent engineers, contributed to make him a very

interesting personage. He was a veritable "treasure trove" on matters connected with the development of railroad construction and science.

In manners, he was well bred, refined, and still simple. He had a keen sense of humor, and his love for young people was a marked trait; for them he always had kind words and cheer.

His whole life, his actions, his works, and his thoughts were full of the "nobility of his soul." He hated shams and pretense, encouraged all elevating influences, was an honor and pride, and a good example to his race, to his nation, and to his profession. Those who knew him well are the better for having known him.

Mr. Haven was elected a Member of the American Society of Civil Engineers on March 5th, 1873.

ARTHUR HAVILAND, M. Am. Soc. C. E.*

DIED JUNE 18TH, 1919.

Arthur Haviland was born at Brooklyn, N. Y., on March 8th, 1848. He was the son of Daniel G. Haviland and Hannah Haviland, who were members of the Society of Friends. His father was one of the originators of Limoges china.

Mr. Haviland received his early education in France and England, and, on his return to the United States, entered Haverford College, at Haverford, Pa., from which he was graduated with the degree of A. B., in 1865.

He began his work as an Engineer in 1869, as Rodman with the San Francisco and Humboldt Bay Railroad, on surveys and construction. From 1869 until 1886, he was engaged on the various phases of railroad engineering—preliminary surveys, location, and construction—in different parts of the United States, including the preliminary and location surveys and maps of the Tehuantepec Interoceanic Railroad and the surveys of the Bar and Harbor of Coatzacoalcos. In 1885 and 1886, Mr. Haviland was employed by Mr. J. D. Crimmins (then President of the Park Board of New York City) as Engineer on his contracting work in New York City.

Leaving the employ of Mr. Crimmins, Mr. Haviland entered the office of the City Engineer at San Antonio, Tex., and, in November, 1886, was promoted to be Assistant City Engineer. In 1887, he returned to New York City on leave to be married.

At this time he accepted a position with the New York Central Railroad Company, in the service of which he remained until his death. While with this Company, he had a prominent part in the depression of the Harlem Division Tracks, between Mott Haven and Woodlawn. Following the completion of this work, he became connected with the Law and Land and Tax Departments, where he pursued lines of work that were of importance and value, and, in that connection, his death will prove to be a severe loss to the Company.

In the course of his work he formed a wide acquaintance with the contents of most of the offices of public records in New York State, and with the public officials attached thereto. He testified as witness for the Company in practically all its important lawsuits, and became known to the judges and the attorneys for his probity, accuracy, and reliance.

Since his marriage, Mr. Haviland had resided in the Borough of the Bronx, New York City, and interested himself locally in many social and civic affairs. He was a member of St. Edmund's Protestant Episcopal Church, and had been deeply interested in all the activities

* Memoir prepared by Henry Vier, Esq., New York City.

connected therewith, having served as a member and Clerk of the Vestry and as one of its Wardens. The large memorial window in the front of the church was presented by Mr. Haviland.

He was always of a cheerful and optimistic disposition, and his loss will be deeply felt by all those who were associated with him intimately, both in business and social life. He is survived by his wife.

Mr. Haviland was elected a Junior of the American Society of Civil Engineers on January 4th, 1882, and a Member on January 4th, 1888.

JOHN VOSE HAZEN, M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1919.

John Vose Hazen was born at Royalston, Mass., on November 22d, 1850. Later, the family resided in Atkinson, N. H., and he was prepared for college at Atkinson Academy. He was graduated from the Chandler Scientific Department (as it was then known), in 1875 with the degree of B. S., and from the Thayer School of Civil Engineering in 1876, with the C. E. degree. The College gave him the A. M. degree in 1908.

From September to December, 1876, Mr. Hazen was employed as Rodman on the Manchester and Keene Railroad, and then served as Principal of the High School at Hancock, N. H., until April, 1877. From April to July, 1877, he was Draftsman for the Boston Bridge Works, and from August, 1877, to July, 1878, Principal of Atkinson Academy. From September, 1878, he served for two years as Tutor in Mathematics and Surveying in the Chandler Scientific School, and then as Professor of Civil Engineering and Mechanics until 1894. From that time until 1911, he was Woodman Professor of Civil Engineering and Graphics in the Scientific Department of Dartmouth College, as re-organized, and also Professor of Civil Engineering in the Thayer School of Civil Engineering.

In the summer season of 1881, Professor Hazen was engaged for three months on a survey of the Connecticut River, at Sumner's Falls, Plainfield, N. H., for a proposed dam. In the summer of 1891, he acted as Chief of Party on the survey of the preliminary line and location of the Hoosac Tunnel and Wilmington (Vt.) Railroad. In the summer of 1900, he was engaged as Chief of Party, for three months, conducting a survey for a proposed electric railroad from Ely Station, Vt., to the Copperfield Mines, in the interest of the late George Westinghouse, M. Am. Soc. C. E., who then owned the copper mines.

From 1900 to 1910, Professor Hazen served as Chairman of the Board of Commissioners of the Village Precinct of Hanover, N. H. He devoted himself diligently to the duties of this office which afforded small emolument and little thanks. He designed and supervised the construction of sewers and drains, and gave much attention to the construction of better roads, building short stretches of good macadam roadway and improving those parts most liable to deterioration. In 1909-10, he was engaged as Expert in the design and layout for an extensive coal-tar-concrete street pavement, in Lebanon, N. H. In 1910, he served his town as Expert in the grading of, and also super-

* Memoir prepared by Robert Fletcher, M. Am. Soc. C. E.

vised the construction of that part of, the New Hampshire State Highway, "West Side Trunk Line", within the town limits. Professor Hazen was always particularly interested in road construction, and labored to secure the passage, by the New Hampshire Legislature, of a general law to establish a State Highway Department which would control road construction throughout the State, thus giving a more efficient and economical administration. The advent of the automobile hastened the result which he so much desired. During the last twenty years, he did considerable expert surveying in Court cases. Being rather modest and undemonstrative in manner, many did not appreciate his ability; but those who consulted him on professional questions found him well-informed, well acquainted with sources of information, and ready to apply his knowledge to the case at hand.

In 1889, Professor Hazen attended the joint conventions of the American Societies of Civil and Mechanical Engineers, the Institution of Civil Engineers, and the French Society of Engineers, in London and Paris. He afterward traveled for a few weeks in Switzerland and England.

As a teacher, Professor Hazen was painstaking and faithful, and had the high respect and regard of his students. He was more particularly interested in descriptive geometry, railroad surveying, highway construction, and foundations. At a time when the means of pictorial presentation of his subjects were not as available as at present, he prepared many large-scale drawings and diagrams to elucidate his lectures and classroom demonstrations. During many years, he directed the preliminary course in Surveying given to Juniors in the College. At the same time, he had charge of the Railroad Surveying and instruction in Railroad Construction given to the first-year class in the Thayer School of Civil Engineering. If all the lines which he projected had been built, Hanover would be a great "railroad center". Among the young men of his classes, Professor Hazen's "stride" was proverbial. In the routine field work few could get over more ground than he, so rapidly, and with, apparently, so little effort. The railroad surveys occupied entire days, the parties taking along their lunches, and it was a rare man who, at the end of the day, would show so little evidence of fatigue as Professor Hazen. There were few, if any, instructors or professors who conducted more class work than he. There were times when he spent from 28 to 30 hours per week in the classroom, including some two-hour exercises and work with students of the Thayer School. Sometimes, with very large classes, he had to get along without as much help from assistants as he would have wished. However, he seldom grumbled, although he was overburdened, but often said that, at the worst, professors and most of the instructors in college have an easy life, free from much of the anxiety and hard

toil which is the lot of the average man. Always ready for duty, he cheerfully accepted the tasks which came to him, and accomplished what some would have regarded as much more than his share.

Professor Hazen died at his home in Hanover, on October 2d, 1919, after suffering from a disability which had gradually increased from early in the summer. He was so well known to successive classes of students through more than 40 years that his passing beyond must awaken the deep sympathy of many who will share in the grief of his family and immediate colleagues.

Professor Hazen was married to Miss Harriet A. Hurlbut, of Hanover, N. H., and his family of two sons and two daughters exemplified the best ideals of home life and neighborly relations. They have, in the recollections of his accomplished lifework, an abiding and precious heritage. He was a member and a regular attendant of the Congregational Church. He had been a member of the School Board for eighteen years, and Treasurer of the Dartmouth Religious Society for many years. Professor Hazen set an example of unflagging devotion to duty and readiness to discharge every responsibility as a citizen, exemplifying the familiar maxim of Smeaton: "The abilities of the individual are a debt due to the common stock of public well being". He had many friends and no enemies, and has left a memory to be cherished by his family and his neighbors.

Professor Hazen was elected an Associate of the American Society of Civil Engineers on June 5th, 1889, and a Member on May 2d, 1911.

GEORGE BLAGDEN HAZLEHURST, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1919.

George Blagden Hazlehurst, the son of Henry R. Hazlehurst and Elizabeth (McKim) Hazlehurst, was born at Baltimore, Md., on November 28th, 1855. He received his education at St. Clement's School, Ellicott City, Md., at Trinity College, Hartford, Conn., and at the University of Virginia where he studied engineering.

After leaving college in 1877, Mr. Hazlehurst was engaged by the Vulcan Iron Works of Baltimore, Md., as Draftsman, Inspector, etc., on shop work on engines, boilers, bridges, etc., and remained with this company until 1881.

In 1881, he entered the employ of the Baltimore and Ohio Railroad Company, serving successively, as Rodman, Topographer, and in charge of a party on preliminary surveys and location until 1882; as Resident Engineer on the Valley Railroad Extension, from Staunton to Lexington, Va., in 1882 and 1883; as Assistant to the Consulting Engineer of the Company in the design and preparation of bridge plans, from 1883 to 1886; as Assistant Engineer in charge of the design and construction of bridges, from 1886 to 1889; and as General Superintendent of Motive Power from 1889 to 1896.

In 1897, Mr. Hazlehurst engaged in private practice as a Consulting Engineer at Baltimore and Catonsville, Md., until 1915, when he was obliged to retire from active work on account of ill health. He died at his home in Catonsville, Md., on November 27th, 1919, after a long illness.

He was married on October 15th, 1890, to Charlotte Gill, of Baltimore, Md., who, with a son and a daughter, survives him.

Mr. Hazlehurst was a man of great reserve and dignity, with most courteous manners. He was always kind and fair to those under him and was held in great respect by all with whom he came in contact. He was a great reader and always kept in touch with current technical and general subjects.

Mr. Hazlehurst was elected a Member of the American Society of Civil Engineers on February 1st, 1888.

* Memoir prepared from information on file at the Headquarters of the Society.

JAMES NISBET HAZLEHURST, M. Am. Soc. C. E.*

DIED FEBRUARY 9TH, 1919.

James Nisbet Hazlehurst was born at Sparta, Ga., on January 15th, 1864. His father, George Hall Hazlehurst, served as a Colonel of Engineers in the Civil War, and was the owner of the Chattanooga Water-Works, and also Chief Engineer of the Macon and Brunswick Railway.

James Nisbet Hazlehurst received his education in the public schools and at Kirkwood Military Academy, and studied civil engineering for two years at the University of the South, at Sewanee, Tenn.

In 1878, at the age of 14, the boy was employed as a Rodman on the extension of the Memphis and Charleston Railway. In 1879, he was engaged, in the same capacity, on surveys for the Texas and Pacific Railway and on the construction of the New Orleans Pacific Railway. In the spring of 1880, he was promoted to the position of Assistant Resident Engineer of the Shreveport Division of the New Orleans Pacific Railway, and in the summer and fall of that year and the winter of 1881, he served as Assistant Engineer and Division Superintendent on the construction of that road from Bayou La Fourche to Provençal, La.

Later in 1881, Mr. Hazlehurst was employed as Assistant Engineer and Superintendent for A. J. Lane and Company, Contractors, on the construction of 120 miles of the Pensacola and Atlantic Railway. In 1882 he went to the New Orleans and Mississippi Valley Railway as Superintendent in charge of the extension from Vicksburg to Port Gibson, Miss., and from Baton Rouge, La., to the Mississippi State Line. He spent the fall and winter of 1884-85 as a student at the "School of Design", in Cincinnati, Ohio, and, from 1885 to 1887, was General Manager of the Lookout Iron Works and Director of the Lookout Water Company, Chattanooga, Tenn., as well as of the Montgomery Water-Works, Montgomery, Ala.

In 1887 Mr. Hazlehurst was Principal Assistant Engineer on location for the Macon and Dublin Railway, but, in 1888, he engaged in private practice as senior member of the engineering firm of Hazlehurst and Hendricks, at Macon, Ga. In 1890 he returned to railway engineering as Chief Engineer on location and construction, and, later, as General Superintendent and Engineer, of the St. Augustine and North Beach Railroad. During 1892 and 1893 he acted as Receiver for the Macon, Ga., Brewing Company, and, during 1893 and 1894, as General Superintendent and Treasurer of the Acme Brewing Company. At this time the Board of Public Works, of Macon, Ga., of which Mr. Hazlehurst was a Charter Member, was organized.

* Memoir prepared by Park A. Dallis, M. Am. Soc. C. E.

In the fall of 1894, he was employed as Constructing Engineer by Gaboury and Noble, Contractors, on sewers for Macon, Ga., and retained that position until the spring of 1895, when he again engaged in the private practice of engineering as the senior member of the firm of Hazlehurst and Company, General Contractors and Engineers, with offices at Atlanta, Ga. This firm contracted for all the sewer work done in Atlanta in 1895, and constructed the sewers and water mains for the Atlanta International and Cotton Exposition. Mr. Hazlehurst also acted during 1896 and 1897 as Consulting Engineer on the design and construction of water systems for Clarksville, Tenn., New Orleans and Algiers, La., Morristown, Tenn., St. Augustine, Fla., Forsyth, Ga., Grenada, Miss., Dyersburg, Tenn., and Hazlehurst, Miss. He also acted as Consulting Engineer to Parrott, Paterson and Company.

In the winter of 1897, Mr. Hazlehurst was appointed Acting City Engineer of St. Augustine, Fla., and completed the water-works of that city and paved most of its streets. In 1898 he was appointed Chief Engineer of the Board of Public Works of Tampa, Fla. In this capacity he located and constructed a complete system of sewers, grading, pavements, and other public improvements. He also served as City Engineer of Tampa.

In 1900 he went to New Orleans, La., as Chief Engineer of the Miles Salt Company, but, in 1901, he removed to Mobile, Ala., where he was appointed Chief Engineer of the Board of Public Works, which position he retained for five years. In 1906 he returned to Atlanta, Ga., opened an office as a Consulting Engineer, and became prominently identified with the engineering activities of that city. He was widely known throughout the South, through his connection as Consulting Engineer for many cities, among which were La Grange, Ga., Raleigh, and Wilmington, N. C., Miami, Key West, Lakeland, and Dunnellon, Fla., and Mobile, Ala. He also made appraisals of water-works for Macon, Ga., Raleigh, N. C., Chattanooga, Tenn., Ocala, Fla., etc.

When the United States entered the World War, Mr. Hazlehurst immediately offered his services to the Government. He was commissioned a Major of Engineers on June 20th, 1917, and assigned to the staff of Maj.-Gen. Leonard Wood, U. S. A., Commander of the Department of the Southeast, as Officer in charge of the Water Supply Section for the camps in the Southeastern States. In September, 1917, Maj. Hazlehurst, with other engineers, was ordered overseas in response to Gen. Pershing's request for expert engineers, and was made Principal Assistant to the officer in charge of the Water Supply Section of the American Expeditionary Forces at Chaumont, France. Later, he was attached to the Third British Army, as an observer in the Valley of the Somme, and was afterward transferred to Tours as Principal

Assistant to the officer in charge of the Water Supply Section of the American forces at that place.

In October, 1918, Maj. Hazlehurst was ordered to the South of France, as Engineer Officer in charge of construction and as Water Supply Officer for Army Base No. 6, his territory extending from the Riviera to the Italian frontier. Later, he was transferred to Paris as a member of the American Peace Commission, and was assigned to the Economical and Financial Section of the Belgian Commission, the duty of which was to study and check the damage due to the German occupation. Maj. Hazlehurst's particular studies were mines, metallurgy, railways, and industrial and general buildings, and he was made Director of the Division having to do with damages to buildings in Brussels, Belgium. While on this duty he was taken ill and died in that city on February 9th, 1919.

Maj. Hazlehurst was one of the foremost engineers of the South, and was prominently identified with the engineering activities of Chattanooga, Tenn., and Atlanta, Ga. As a resident of the latter city, he took a leading part in the organization of the Affiliated Technical Societies of that place, the first organization of its kind in the United States. He also served as Chairman of the Committee on Engineering and Good Roads of the Atlanta Chamber of Commerce, was a member of the Commission which selected the site of Camp Gordon as a military cantonment, and a member of the National Council of Defense. Maj. Hazlehurst was also a member of the American Waterworks Association and the American Public Health Association. He had written a number of articles for various technical publications and was the author of a book entitled, "Towers and Tanks."

He was married in 1886 to Miss Mary Griffiss, of Chattanooga, Tenn., who, with two sons, George Hall Hazlehurst, Sanitary Engineer of the State of Alabama, Montgomery, Ala., and Capt. John G. Hazlehurst, U. S. A., survives him.

Mr. Hazlehurst was elected a Member of the American Society of Civil Engineers on October 4th, 1899. He was a Charter Member and the first Chairman of the Executive Committee of the Atlanta Association of Members of the American Society of Civil Engineers.

JOHN GIBSON HENDRIE, M. Am. Soc. C. E.*

DIED JUNE 18TH, 1919.

John Gibson Hendrie, the son of Daniel and Jessie Gibson Hendrie, was born on December 17th, 1881, at Philadelphia, Pa.

He received his early education in Philadelphia, attending the Central High School from 1897 to 1901. He entered the University of Pennsylvania in 1902, where he took a four-year course in Civil Engineering, which he completed in June, 1906.

Mr. Hendrie gained his early experience as a Civil Engineer, with the Trussed Concrete Steel Company, the Pennsylvania Railroad Company, Franklin and Clarke, and McCormick and Company, respectively. For the last-mentioned firm, he acted as Resident Engineer in charge of the engineering work on the construction of the Mulberry Street Bridge at Harrisburg, Pa.

He served as Engineer for Weeks and Sanville until 1911, when he went with the Barber Asphalt Paving Company, and was located at Maurer, N. J., in charge of construction work until 1913. Mr. Hendrie was then sent to Brighton, Trinidad, as Engineer in charge of the Asphalt Works at that place. He returned to Maurer in 1916, and remained there until he resigned to enter the Engineers Corps of the Army.

He was commissioned a Captain in the Reserve Corps on August 15th, 1917, and served with that rank from September, 1917, until July 30th, 1918, when he was commissioned a Major.

Maj. Hendrie served in France until he was detached and ordered back to the United States to assist in the training and development of a new battalion of Engineers. He resigned from the service in December, 1918, and went to Fort Worth, Tex., as Engineer with the Pierce Oil Corporation, where he remained until his death on June 18th, 1919.

Of Scotch descent, Maj. Hendrie inherited the best Scottish traits and characteristics. He was very active in football, rowing, and other undergraduate activities during his high school and college courses. He was a member of the Phi Delta Theta Fraternity and was elected Cane Man of his class in college.

He was married on April 26th, 1913, to Maria Elizabeth Roessler, of Perth Amboy, N. J., who, with three children—Donald, Elizabeth Roessler, and John Gibson—survives him.

Maj. Hendrie was elected a Junior of the American Society of Civil Engineers on September 5th, 1911, an Associate Member on April 2d, 1913, and a Member on May 12th, 1919.

* Memoir prepared by George A. Huggins, Esq., Philadelphia, Pa.

HENRY WILSON HODGE, M. Am. Soc. C. E.*

DIED DECEMBER 21ST, 1919.

When a man passes into the next life, leaving behind him a record of high accomplishment, the World seeks for the causes that lifted him above the general level of his fellows and in so doing gives first place to those influences that play so large a part in the moulding of man's character and in the shaping of his destiny—heritage and family tradition, early training and environment, education, and the associations of later years.

In the life of Henry Wilson Hodge the effect of these influences was manifest to a remarkable degree, in cultivating the qualities that won for him lasting fame, not only as a brilliant engineer, but in an equal degree as a patriotic citizen, a loyal friend, and a fearless man of broad humanities and lofty ideals to which he was ever true.

Henry Wilson Hodge was born in Washington, D. C., on April 14th, 1865—the day made memorable by the assassination of President Lincoln—of a long line of American ancestry reaching back to early Colonial times.

His father, John Ledyard Hodge, a great-grandson of the first of the line in America, who came from Ireland about 1745, was born in Philadelphia, Pa., was graduated from Princeton, and removed to Washington at the time that his father, William Ledyard Hodge, was made Assistant Secretary of the Treasury during the administration of President Fillmore. The paternal grandmother of Col. Hodge was Sarah Bayard, of Philadelphia, whose grandfather, John Bayard, was a leading man in that city and as Colonel of a Philadelphia regiment during the Revolutionary War served in many actions and was personally complimented by Gen. Washington for his gallantry at the Battle of Princeton. This John Bayard was a great-grandson of Peter Bayard, who came to this country in 1647 with his uncle, Peter Stuyvesant, the last Dutch Governor of New Amsterdam.

The mother of Col. Hodge was Susan Savage Wilson, a daughter of Henry Parke Custis Wilson, of Virginia, and a sister of the eminent surgeon of that name in Baltimore, Md. This family, which was of Scotch-Irish extraction, settled in America in 1700 and was instrumental in founding the first Presbyterian Church in this country at Rehoboth, Md. Mrs. Hodge was a deeply religious woman of noble character, and her son often remarked to his friends that, raised as he was under Spartan conditions with no surplus of worldly possessions,

* Memoir prepared by the following committee: W. J. Wilgus, Frederic A Molitor, Howard C. Baird, and Robert Giles, Members, Am. Soc. C. E.

it was to her teachings and example that he owed the best in his character.

In fact, it is hard to imagine how Mr. Hodge could have been otherwise than a man of high principle, with such fine traditions, early training, and environment.

Educated at Young's Private School in Washington, his health, always delicate as a child, broke under the strain of untiring study, and at the age of fifteen he joined a surveying party on the Chesapeake and Ohio Railroad in the mountains of West Virginia, and carried a chain for nearly two years. The experiences of this rough outdoor life not only built up his constitution, but awakened in him the ambition to become an engineer. Acting on this thought he borrowed the necessary money through the kindness of a relative, and, in 1882, entered Rensselaer Polytechnic Institute, from which he was graduated in 1885.

Now fairly launched in his life's work, Mr. Hodge entered the service of the Phœnix Bridge Company, with which for six years, under the direction of its Chief Engineer, the late A. Bonzano, M. Am. Soc. C. E., he was successively Draftsman and Computer, Assistant Engineer and Resident Engineer, gaining through practical experience the broad foundation on which was reared his future success. Among the structures with the design of which he was connected during this period was the Huntington Bridge at Cincinnati, Ohio. He also had much to do with the planning and erection of the Pecos Valley Viaduct of the Southern Pacific Railway.

In 1891 came an important change in his fortunes when he accepted the position as Chief Engineer of the Union Iron Works, at New York City, in which city he spent the remainder of his life.

About this time Mr. Hodge joined the University Place Presbyterian Church and shortly after became an Elder and, later, one of its Trustees, giving freely of his time and substance to a cause which was always dear to him. For nearly thirty years, he was Superintendent of the Sunday-school of this Church, which he attended with unfailing regularity and where he exerted a great influence over hundreds of young people.

Soon after coming to New York City he had joined the Seventh Regiment of the National Guard. A stranger in the city, he found his service in Company K a real pleasure, and the friendships which he made there were among the warmest of his life. He always regarded his old regiment with genuine affection.

An important step in his life came in 1893 when his employment with the Union Iron Works was terminated, and he ventured into the field of the independent consulting engineer. For the ensuing two years Mr. Hodge did much important work, one instance of which was

the preparation of standard plans of bridges, of lengths ranging from 10 up to 200 ft., for the 900-mile extensions of the Choctaw, Oklahoma and Gulf Railroad, of which Frederic Molitor, M. Am. Soc. C. E., was then Chief Engineer.

In 1895 he entered the employ of the late Alfred P. Boller, M. Am. Soc. C. E., with whom four years later he entered into partnership under the firm name of Boller and Hodge, afterward changed, through the admission of Howard C. Baird, M. Am. Soc. C. E., to Boller, Hodge and Baird. Following the death of Mr. Boller in 1912, the firm name was changed to Hodge and Baird.

During this period of a quarter of a century the work for which Mr. Hodge was largely responsible covered a wide field, embracing noteworthy structures in many parts of the world. Among these were the sightly Great Northern Railroad Bridge, at Duluth, Minn.; the Choctaw, Oklahoma and Gulf Railroad Bridge, at Little Rock, Ark., the pleasing design of which stands as a monument to his instinct for good taste; all bridges for the National Railroad Company of Mexico and for several South American companies, among which were the Brazil, Cerro de Pasco, Bolivian Central, and Madeira-Mamoré Railways; all bridges for the Philippine Railways and Manila Railway; the Riverside Viaduct at 96th Street and the Melrose Avenue Viaduct, in New York City; the bridges over the Connecticut River at Saybrook and East Haddam; and all bridges for the Wabash-Pittsburg Terminal Company, including the impressive cantilever bridges of that company over the Ohio and Monongahela Rivers, the latter having a span of 812 ft., which was the longest railroad span then in existence with the exception of the Forth Bridge. His crowning work was the great Municipal Bridge over the Mississippi River, at St. Louis, Mo., in which his initiative was notably displayed in the use of nickel steel for the main members of the exceptionally long simple channel spans.

In an advisory or consulting capacity Mr. Hodge's genius was utilized on the Blackwell's Island and Manhattan East River Bridges of the City of New York; the rehabilitation of the elevated lines of the Brooklyn Rapid Transit Company; the rebuilding of the Quebec Bridge after the collapse of the original structure; the Singer and Metropolitan Towers, in New York City; and studies and investigations for the proposed great interstate bridge over the North River at New York City. He was, at the time of his death, a member of the Board of Consulting Engineers for the proposed vehicular tunnel under the North River, between Lower Manhattan and Jersey City, N. J.

It is not only by this multitude of examples of his engineering skill that the professional work of Mr. Hodge is to be judged. Never seeking office, it was with reluctance that he consented, through a sense of duty and at great personal sacrifice, to accept a place on the Public

Service Commission of the New York District proffered to him by Governor Whitman in January, 1916. His peculiar function as a Commissioner was to supervise the construction of the great subway extensions then in progress, as well as to deal with many matters of a regulatory nature. In this trying place he revealed fearlessness and powers of decision which won for him the renewed admiration of his friends and the respect and support of his colleagues, the press, and the community at large. His remarkable success as a Commissioner went far to support the claim that the engineer of character, experience, and administrative capacity is ideally suited for high public office.

The entry of the United States into the World War brought a ready response from Mr. Hodge, who sometime before had patriotically volunteered for and received a commission in the Engineer Officers' Reserve Corps. Governor Whitman felt that the subway situation in New York City was such that Mr. Hodge owed his first duty to the State, but, later, he withdrew his opposition to and accepted the resignation of Mr. Hodge, who sailed for France with the rank of Major, in July, 1917.

From this time until his discharge Col. Hodge rendered notable service to his country, first in charge of the design of railroad structures for the rapidly expanding transportation needs of the Army in France, then as Manager of Roads, and, finally, as Assistant Chief Engineer of the Army in charge of military bridges. He was promoted to the rank of Lieutenant-Colonel on October 26th, 1917, and to Colonel on August 13th, 1918, and received his honorable discharge on January 22d, 1919, soon after his return to the United States.

In the months that followed Col. Hodge entered vigorously into the rebuilding of his professional practice, but his friends noticed a change for the worse in his physical condition, beyond doubt brought about by his experiences in the Army. His sudden death, however, on December 21st, 1919, came as an unexpected shock to them.

Col. Hodge was a man of commanding figure, tall and well proportioned, with features that reflected a rare combination of strength and kindliness. His manners were always courteous and marked with the frankness and sincerity that command confidence and respect. Humor to an unusual degree was inherent in his nature and smoothed not only his own path through life but also the rugged roads of his friends, who never looked to him in vain for warm sympathy and helpful advice. To quote the words of Tennyson he was:

> "That tower of strength
> Which stood four-square to all the winds that blew."

Mentally, spiritually, physically, he truly was a man of mark, loved for his warm humanity and lofty ideals, admired for his commanding

ability and strength of purpose, and respected for his straight thinking, sterling integrity, and fidelity to duty.

Col. Hodge was married on December 14th, 1897, to Sarah Cunningham Mills, only child of George Mills, of Savannah, Ga., who survives him.

He was a Trustee of Rensselaer Polytechnic Institute, from which he received the degree of Doctor of Engineering in 1918, a member of the Council of New York University, a Director of Princeton Theological Seminary, and a Director of the American Church in Paris. He was also a member of the Board of Foreign Missions of the Presbyterian Church and of the Board of Managers of the Knickerbocker Hospital of New York City, and a Governor of "Chester Crest", Mount Vernon, a home for intemperate men.

In his profession, he was a member of the American Institute of Consulting Engineers, in which he was a Past Director, the Canadian Society of Civil Engineers and the Institution of Civil Engineers of Great Britain. He was also a Veteran of the Seventh Regiment, and Secretary of the Associates of the Engineer Corps of Company K, Seventh Regiment, National Guard, New York; a member of the Citizens' Union of New York City; and a member of the Chamber of Commerce of the State of New York. His clubs were the Century, University, and Downtown of New York City.

Col. Hodge was elected a Junior of the American Society of Civil Engineers on January 5th, 1887, and a Member on October 2d, 1895. He served as a Director of the Society from 1913 to 1915, inclusive.

FRANK CHURCHILL HORN, M. Am. Soc. C. E.*

DIED FEBRUARY 16TH, 1919.

Frank Churchill Horn, the son of Adam E. and Martha P. Horn, was born at Macomb, Ill., on January 8th, 1861. He was educated in the public schools and under private instructors.

His first engineering engagement, after acquiring some experience as assistant to an engineer in private practice, was in 1882 as Assistant Engineer on the construction of the St. Louis and San Francisco Railroad in Western Arkansas. In 1884, he was engaged as Assistant Engineer on railway location, in Iowa, Minnesota, and Dakota, for the Burlington, Cedar Rapids, and Northern Railway.

From 1885 to 1889, Mr. Horn was Principal Assistant Engineer, and, later, Chief Engineer and Superintendent, of the Department of Public Works of Lake, Ill. During this period he remodeled and enlarged the sewerage and water-works systems, besides making many other improvements in the town.

From 1889 to 1894, he served as Engineer and Superintendent for a number of contracting firms in the construction of water-works and sewerage systems in Indiana, Alabama, Ohio, and Illinois, and also for the construction of the Intramural Railway for the World's Columbian Exposition at Chicago.

In the autumn of 1894 he was engaged by the Lake Shore and Michigan Southern Railway on track elevation in Chicago. In 1895 he was employed by the Sanitary District of Chicago to construct the regulating works of the Chicago Drainage Canal at Lockport, Ill., and, again, in 1898-99, he was placed in charge of building masonry structures and bridges on the Drainage Canal. In 1900 he was employed by the Economy Light and Power Company on the construction of the hydro-electric plant at Joliet, Ill.

In September, 1900, Mr. Horn came to Idaho as Assistant Chief Engineer on the construction of the Swan Falls Hydro-Electric Plant on Snake River. On the completion of this work in 1901, he became Resident Engineer of the Denver Union Water Company, of Denver, Colo., in charge of the construction of the Platte River Canyon filtration plant and afterward of the Lake Cheesman Dam which forms a reservoir for the storage of the Platte River water for the City of Denver.

In 1905 Mr. Horn was appointed Construction Engineer in the United States Reclamation Service, and came to Idaho as Construction Engineer, first for the Minidoka rock-fill dam on Snake River, then

* Memoir prepared by A. J. Wiley, M. Am. Soc. C. E.

for the Boise River concrete diversion dam and the Upper and Lower Deer Flat earth-fill embankments.

Resigning from the Reclamation Service in 1908, he became Construction Engineer for the Salmon River Dam of the Twin Falls Salmon River Land and Water Company, near Twin Falls, Idaho. On the completion of this work in 1911, Mr. Horn opened an office in Boise, Idaho, as Consulting Engineer, and continued in this practice for the remainder of his life. In connection with the lining of the tunnels at the Salmon River Dam, he invented a process for the pneumatic conveyance and placing of concrete and devoted much of his energy to developing this process and putting it in commercial use. Among the most successful examples of its application were the lining of the Port Richmond and Stockton Street Tunnels in San Francisco.

Mr. Horn's most prominent characteristic as an engineer was the uncompromising thoroughness of his work. This was shown very strongly on the Lake Cheesman Dam, which is one of the very few, if not the only, rubble masonry dam which, from its first construction, was entirely free from any trace of seepage or sweating.

From his early experience in contracting, he acquired a taste for the personal conduct of construction work, and was always a strong advocate of force-account construction as opposed to contracting for the work under his charge. The Upper Deer Flat Embankment of the Deer Flat Reservoir of the United States Reclamation Service in Idaho is an earth dam containing more than 1 000 000 cu. yd. This dam was built by Mr. Horn with Government forces and equipment at a saving of more than $100 000 below the lowest bid. The Salmon River Dam, containing about 150 000 cu. yd. of concrete and 3 600 ft. of outlet tunnel, was also done under his direction by force account at a very low cost. He was an expert in all classes of construction machinery, and was employed by the United States Reclamation Service as Consulting Engineer in the selection of the construction equipment for the Arrowrock Dam in Idaho.

Mr. Horn was intensely loyal to his associates, and had their unqualified confidence, affection, and respect. He was an unselfish and devoted friend, and few men stood higher in the community where he spent the last years of his life. Though not a college graduate, he was elected a member of the University Club of Boise, and was its Vice-President at the time of his death. He was very much interested in questions of public health and, at one time, paid the hospital fees for the public school children of Boise, whose circumstances would otherwise have prevented them from having a necessary operation.

He was married on June 24th, 1888, to Margaret K. Ferguson, of Guelph, Ont., Canada, who died in 1899, leaving no children. His

intense devotion to the memory of his wife was one of the most touching of his many noble and lovable traits.

He died in Boise, as the result of a stroke of apoplexy, on February 16th, 1919. He is survived by a sister and two brothers, and leaves many friends and associates among whom his memory will always be cherished.

Mr. Horn was elected a Member of the American Society of Civil Engineers on September 1st, 1903.

RUFUS CAMERON HUNT, M. Am. Soc. C. E.*

DIED FEBRUARY 18TH, 1919.

Rufus Cameron Hunt was born at Bowling Green, Ky., on December 1st, 1851. His father was a farmer and long a resident of the State. His early education was obtained at a local school kept by a Professor Shields. Without the advantage of a college education, he taught himself the rudiments of engineering and began his life's work in railway location and construction—a great field, but one from which only the efficient rise to prominence or a competence.

In 1898 Mr. Hunt was a Division Engineer on the construction of a railway in Maine without apparent prospects, but his careful, conscientious work had attracted the attention of those who are always on the lookout for capable men. In that year the Bryce Syndicate was organized, under the name of the American China Development Company, for the construction of railways in China, and Mr. Hunt was offered a position as Assistant Engineer. Later, he was made Locating Engineer, and had charge of the survey from Hankow to Canton, 750 miles. Part of the line has been constructed, as laid out by him. He remained in China for some time as Chief Engineer. From China, Mr. Hunt went to Europe, and thence for more than a year to South America where he had charge of important work in the Andes Mountains and Peru. His next field was Mexico.

In the meanwhile, the subway in New York City was under construction, and the contractor who had charge of the Fort Washington Tunnel, from 160th to 196th Streets, had become so much involved that the bonding company had to take over the work. As the result of a conference between the Company, the General Contractor, and the Chief Engineer of the Commission, Mr. Hunt was selected to take charge of this highly important and difficult section. So well did he carry out his trust, that Mr. John B. McDonald subsequently placed him in charge of the Jerome Park Reservoir contract. Then followed other large pieces of contract work, but, on their completion, Mr. Hunt decided to retire from the field where he had succeeded so well and take up the life of his forefathers, that of a country gentleman. To this end, he acquired an extensive farm near Washington Court House, in Fayette County, Ohio, with which he had the same success as he had building railways or digging tunnels. He died at his home on February 18th, 1919, of pneumonia following an attack of influenza. He is survived by his widow, who was Miss Lida Saxton.

His character was like his mind and frame, rugged and strong. It was marked by a rare combination of firmness and justice, but, if any

* Memoir prepared by William Barclay Parsons, M. Am. Soc. C. E.

one quality predominated, it was unwavering loyalty. He loved truth, hated shams, and was a fine specimen of a real American. These qualities endeared him to all. As an engineer he had sound judgment which, combined with his personal traits, made him a real leader of men.

Mr. Hunt was elected a Member of the American Society of Civil Engineers on October 5th, 1904.

LINN MURDOCH HUNTINGTON, M. Am. Soc. C. E.*

DIED NOVEMBER 24TH, 1919.

Linn Murdoch Huntington was born on November 11th, 1884, at North Bend, Nebr., and was descended from old New England stock. He was graduated in Civil Engineering from the University of Nebraska in 1904, having spent all his vacations in railroad work.

After his graduation, Mr. Huntington went almost immediately to the Panama Canal and spent practically the remainder of his life in the tropics. During his three years in Panama, he rose rapidly from Rodman to Assistant Superintendent of Public Works and Assistant Engineer of Maintenance of the Panama Railroad. Transferred to Cuba during the Second Intervention, at the request of Governor Magoon, he became Second Engineer in the Department of Public Works and, later, was engaged in consulting work with Marx and Windsor, an American contracting firm.

Leaving Cuba in 1910 Mr. Huntington joined the Department of Public Works of the Dominican Republic, which he served most faithfully and creditably in many capacities until 1913, when nearly all the American employees were precipitately discharged from the service.

Shortly afterward, in 1914, he became Vice-President and General Manager of the Bramon Estates Company, which owned and operated one of the largest coffee plantations in Venezuela, where he remained until a few months before his death. It was while on this work that his wonderful persistence and splendid loyalty showed to its best advantage. For more than a year he stayed on, fighting an unequal battle for the retention of the property fraudulently taken over by German interests, though he received no salary and little support from his principals at home.

Mr. Huntington was a pre-eminent example of the fighting American engineer who has had such a strong influence in every land where the empire builder has followed the explorer into the out-of-the-way places of the earth. He might have been the inspiration of the "Soldier of Fortune" which has enticed so many of his kind to the land of the *dolce far niente*.

"Hunt" was a typical "soldier", fighting and working incessantly, never quitting, neither stopping nor giving up till the job was done, whatever it might be. Obstacles added zest to the work, but he never dreamed of failure. At the end it was the same; though suffering from a disease of the throat which had prevented him from taking even water and having been kept alive for several days with oxygen, he gave no sign that he despaired of recovery. As far as any one could see, he considered that his prospects were of the brightest, and his plans were

* Memoir prepared by John L. Mann, M. Am. Soc. C. E.

all for the future. He died on November 24th, 1919, and is survived by his widow and a son.

Having spent practically all his professional life in the tropics, Mr. Huntington had absorbed the atmosphere of the Latin American and spoke the language so well that he was frequently taken for a native of various South American countries.

He was a member of the National Arts and Salmagundi Clubs of New York City, and was the author of several short stories published by Harper's and other magazines.

Mr. Huntington was elected a Junior of the American Society of Civil Engineers on May 2d, 1905, an Associate Member on August 31st, 1909, and a Member on December 31st, 1913.

CHARLES MATTATHIAS JACOBS, M. Am. Soc. C. E.*

DIED SEPTEMBER 7TH, 1919.

Charles Mattathias Jacobs was born at Hull, Yorkshire, England, on June 8th, 1850, and died at his home at Laugharne, South Wales, on September 7th, 1919.

He was the ninth in a family of fourteen born to Bethel and Esther Jacobs. His father was a member of a family long established in Hull, a well known and highly honored merchant of that city.

Mr. Jacobs was educated by private tuition and, in 1866, at the age of 16, was articled as a pupil in the arts of mechanical and structural engineering with Messrs. Charles and William Earle, one of the leading ship and engine building firms on the northeast coast of England. With this firm Mr. Jacobs served his five-year term of apprenticeship, obtaining a thorough knowledge of all the trades involved in ship and engine building, as well as in the design of engines and structures.

This pupilage in practical work was then the usual British system of technical training. Throughout his long service as a professional engineer, Mr. Jacobs' practical understanding of, and his judgment in solving, engineering problems always reflected that early training.

In 1871, on the completion of his apprenticeship, he was sent by Messrs. Charles and William Earle to China to put up some bridges which they had contracted to build and erect.

On his return to England, Mr. Jacobs decided to follow the profession of a marine engineer and to that end went to sea in the Merchant Service for between two and three years, the period necessary to obtain the British Board of Trade certificate as First-Class Marine Engineer.

In 1876 he opened an office in Cardiff, South Wales, as a Consulting Engineer, specializing in marine engineering.

In this same year he married Frances H. M., daughter of the late John Fry, of Penarth, a leading colliery operator and shipowner. His widow survives him.

Soon after this, he was appointed Surveyor to Lloyd's Register of Shipping at Cardiff, and he held this office for a few years. During this period he became recognized as an expert and authority in a business in which Cardiff and Newport stood pre-eminent, namely, the briquetting and consequent utilization, of the slack coal-dust from the non-coking coals of South Wales and Monmouthshire.

On December 1st, 1885, he was elected a Member of the Institution of Civil Engineers. In 1887, Mr. Jacobs removed to London, where he opened a Consulting Engineer's office in partnership with Mr. Herbert Barringer, under the firm name of Jacobs and Barringer. This

* Memoir prepared by J. V. Davies, M. Am. Soc. C. E.

connection, which broadened the scope of his engineering work, continued until his death.

In 1888, the late Austin Corbin, who was then President of the Long Island, the Philadelphia and Reading, and the Elmira, Cortlandt and Northern Railroads, besides having wide interests in other organizations and subsidiary companies, became deeply interested in the possibility of using the culm or waste produced in the crushing and sorting of anthracite coal in the fields of the Philadelphia and Reading Coal and Iron Company. While in Europe in that year, Mr. Corbin consulted with Mr. Jacobs as to the possibilities of this development and invited him to come to the United States, which he did early in 1889. This was the beginning of the long and close association between Mr. Corbin and Mr. Jacobs, by which Mr. Jacobs became directly identified as Consulting Engineer with all the engineering problems of the various interests controlled by Mr. Corbin.

The most pressing problem at that time was that of an improved terminal for the Long Island Railroad. Mr. Jacobs took up the study of an extension of the Long Island Railroad from its terminals in Brooklyn and Long Island City. In co-operation with Samuel Rea, M. Am. Soc. C. E., of the Pennsylvania Railroad, he developed a plan for the extension of the Brooklyn lines by tunnel under the East and North Rivers, which would give passenger connection direct to the Pennsylvania Railroad Company's Jersey City terminal. One element in these studies was the removal from Atlantic Avenue, Brooklyn, of the surface tracks of the Long Island Railroad. The preliminary studies and estimates for doing this by elevating and depressing the railroad were made under the direction of Mr. Jacobs, and the work was later carried out by his partner, J. V. Davies, M. Am. Soc. C. E., as Chief Engineer to the Board for the Atlantic Avenue Improvement. This work, and the other studies made for the improvement of the Long Island Railroad, initiated Mr. Jacobs' connection with the rapid transit situation in New York City.

In 1891, Mr Jacobs opened an office as Consulting Engineer in New York City, his Chief Assistant being Mr. Davies, with whom he later entered into partnership. The company was finally organized as Jacobs and Davies, Incorporated, and continued without a break until the end of 1916, when Mr. Jacobs retired from practice in the United States.

In 1892, he undertook for the East River Gas Company the design and supervision, as Chief Engineer, of the construction of a tunnel having an internal diameter of 10 ft. 2 in. to connect the Company's plant at Ravenswood, Long Island, with the Borough of Manhattan. This tunnel starts at the foot of Webster Avenue, Long Island City, and ends at the foot of East 71st St., New York City, passing under both channels of the East River and under Blackwells Island. The work was a notable achievement for Mr. Jacobs. Soon after the tunnel had

been started, a seam of rotten rock giving direct access to the river was encountered, the contractor abandoned the work and Mr. Jacobs took it over on behalf of the Company, installed a tunneling shield, and finished it with his own forces after many and serious difficulties had been met and conquered.

This was the first tunnel to be finished under either the North or East Rivers, although others had been begun at an earlier date under both rivers. The air pressure used was very high, reaching 48 lb. per sq. in. (above the normal), and this is believed to be the first case in which extensive rock blasting was done under compressed air.

In 1895, Mr. Jacobs was appointed by the bondholders of the Company owning the then unfinished and abandoned Hudson River Tunnel at New York City, as one of a committee of three to examine and report on the condition of and possibility of finishing that work. As a result of this investigation, and after long delay in re-organizing the finances of the old concern, a new company was formed, under the presidency of Mr. William Gibbs McAdoo, known as the New York and Jersey Railroad Company, which took over the old work and carried on the enterprise. This was the nucleus about which the present Hudson and Manhattan Railway System of passenger railroad service between New York City and Jersey City has been built up and which now acts as the down-town New York distributing terminal of the Pennsylvania Railroad and also as the up-town and down-town New York terminals of the Delaware, Lackawanna and Western Railroad and the Erie Railroad.

The design and construction of this important system of tunnels across the North River and in New York City and Jersey City, involving many unprecedented problems both as regards design and execution, were carried out for the company by Mr. Jacobs as Chief Engineer, with his own designing, engineering, and construction forces. The construction of this system, as it stands to-day, was finished in 1910. By the completion of the "Old North Tunnel", begun in 1874, and dogged by persistent ill luck until Mr. Jacobs took over the work in 1902, the conquest of the North River was achieved, and Mr. Jacobs, of all his successes, always looked back with the greatest pride to the fact that on March 11th, 1904, he was the first man to have crossed, dryshod, beneath the great North River, from New Jersey to New York City. The work of tunneling the North River for this company was described by Mr. Jacobs in a paper* presented to the Institution of Civil Engineers on February 22d, 1910. For this paper, the Council of the Institution gave him its premier award, the Telford Gold Medal.

In 1898, Mr. Jacobs prepared the plans and specifications and awarded the contract for the Tredegar Dry Dock at Newport, Monmouthshire, on the western side of the River Usk. This dry dock is 708

* *Minutes of Proceedings*, Inst. C. E., Vol. CCXXXI (Paper No. 3859).

ft. long and 65 ft. wide at the entrance. It can take a ship of 12 000 tons, and is divisible in two sections by middle gates, so that two ships of 7 000 tons each can be docked at the same time. Many serious difficulties, due to the nature of the ground, were met and successfully overcome and the work* was finished in 1903.

The other great New York project with which Mr. Jacobs' name will be forever linked is that which brings into New York City the terminal of the Pennsylvania Railroad. It is impossible here to do more than mention that Mr. Jacobs was the Chief Engineer of the North River Division of this work, and thus had direct charge of the designs and construction of all that part which lies between the Terminal Station and the Western Portal of the tunnel through the Bergen Hill. The length of the Division is 14 280 ft., all in tunnel and all providing for two lines of track. The material penetrated ranged from the softest silt to the hardest trap rock, with every possible variety lying between the two. The work is described fully in papers by Messrs. Raymond, Jacobs, Noble, Gibbs, and members of the staff.†

Mr. Jacobs' own paper‡ describes fully his personal connection with this work dating from his consultation in London in 1901 with Mr. Cassatt, then the President of the Pennsylvania Railroad. This paper contains also a most interesting sketch of the events which preceded and led to the inception of this great scheme. One thing, however, must be made clear as it is only touched upon most lightly in his own account, and that is, that without the assurance given by Mr. Jacobs' invention to provide support, if such should be found necessary in the North River silt, for the structure and tracks of tunnels, by an internal bridge construction within the tunnels themselves, supported by piers or piles sunk to firm foundations, it is unlikely that the management of the Pennsylvania Railroad would have ventured on the uncertainties of building the great main line tunnels in this silt. Protracted and exhaustive tests during the construction period proved that these supports were not needed, but the value of their conception as an assurance to those who planned this bold feat cannot be over-estimated.

Both these undertakings, involving the crossing of the North River by six different tunnels, were carried out simultaneously by Mr. Jacobs between 1901 and 1910. Many of the problems were literally unprecedented, and to the pioneer solution of these Mr. Jacobs brought his ever practical viewpoint, his convincing personality, his indomitable perseverance, and the keen edge of his enthusiasm.

Many other works of large size and great difficulty have been carried out under Mr. Jacobs' general direction, among which are the following, stated more or less in chronological sequence.

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXVII (Paper No. 3646).

† *Transactions*, Am. Soc. C. E., Vols. LXVIII and LXIX (1910).

‡ "The North River Division," *Transactions*, Am. Soc. C. E., Vol. LXVIII (1910), p. 32.

He was closely identified with the oil industry and built many oil storage installations in Great Britain and elsewhere, one of which was a pipe line for the Burmah Oil Company from Rangoon to Yenangyat. This line consists of about 270 miles of 10-in. and 30 miles of 4-in. pipe, between the wells and the seaboard, together with pumping stations and tank equipment. The surveys were made in 1902 and the installation was finished in 1908.

From 1907-11, he had charge, as Consulting Engineer to Messrs. Daydé and Pillé, Contractors, of the tube tunnels driven by shield under compressed air below the River Seine and the Place de la Concorde, in Paris.

From 1910-15, the construction of the Astoria Tunnel, 4 662 ft. long and 275 ft. below the waters of the East River, was accomplished. This tunnel is used to contain two cast-iron mains, 6 ft. in diameter, by which gas is conveyed from the plant of the Astoria Light, Heat, and Power Company to East 132d Street, The Bronx. This tunnel is 20 by 20 ft. in cross-section and gave rise to what was probably the worst problem in tunneling under normal air pressure yet presented in New York City. This work, which was designed and carried out by Mr. Jacobs' own forces, has been fully described.*

From 1910 to 1913, the construction of the Hales Bar Lock and Dam across the Tennessee River near Chattanooga, Tenn., was carried out by Mr. Jacobs' firm. In this case, in order to obtain secure foundations in a limestone rock, honeycombed with caverns and pockets, very large monolithic caissons had to be sunk under pneumatic pressure.

In 1912-13 the construction of the Laxaxalpam Aqueduct Tunnels of the Mexican Light and Power Company was completed by Mr. Jacobs' firm with its own forces after the work had been at a standstill on certain essential portions, owing to the severe physical difficulties, for more than a year. This work comprised a total length of 20 miles of tunnel in a mountainous part of the State of Puebla, Mexico, and notwithstanding serious physical difficulties, due both to the geological formations and the remote and mountainous nature of the country, was finished within the time desired.†

Mr. Jacobs' practice kept in close touch with marine work to the end, particularly in ship and engine building. He was responsible for the building of some 84 tank steamships and more than 100 cargo and other vessels.

Some of the large projects which he fathered, or with which he was connected, are the following:

The location of a railway by which the Long Island Railroad would cross by tunnel under Long Island Sound and connect with the Housatonic Railroad. This would have given a route to Boston, *via* the New

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 594.

† *Minutes of Proceedings*, Inst. C. E., Vol. CC, p. 345 (Paper No. 4123).

England Railroad, and it was thought would form a rival to the New Haven Railroad System. The absorption of the New England lines by the New Haven Company stopped this project.

At the same time, however, the contemplated scheme necessitated a suitable terminal station in New York City, on Manhattan Island, involving the crossing of the East River, with yards and a terminal adequate for the operation, not only of the local traffic of the Long Island Railroad, but also the through business of this projected line with its connections. Studies were then made as to the feasibility of a tunnel crossing in the vicinity of the existing Long Island Railroad Terminal, at Long Island City, but it then appeared that the cost of such necessary terminals in the East 34th Street district would be so high as to be prohibitive.

At this juncture, Mr. Corbin was offered by the late Dr. Thomas Rainey a franchise he held for a bridge structure to cross the East River between Ravenswood, on Long Island, and the foot of East 64th Street, New York City. After extended negotiations and investigations, this franchise was taken up by interests associated with the Long Island Railroad, and notwithstanding the withdrawal of the New England connection problem, it was considered that the plan of development was necessary to the extension of the Long Island Railroad itself, and, consequently, Mr. Jacobs was instructed to proceed with the construction of the bridge, terminal, and connections. This work was actually and actively commenced. Foundations for three main piers at the bulkhead lines of the two channels of the East River were built, and contracts for the masonry and superstructure were awarded, when the accidental death of Mr. Corbin in 1896 suspended all activities, which were never renewed. The appointment of Mr. W. H. Baldwin, Jr., as President of the Long Island Railroad, re-opened the study of the possibility of a terminal which would connect the Long Island Railroad with the Pennsylvania System, which, at that time, had in contemplation the construction of a North River Bridge entry into New York City. The later history of this development and Mr. Jacobs' connection therewith are stated in the *Transactions* of the Society.

The development of Fort Pond Bay at Montauk Point as a terminal for the trans-Atlantic steamship traffic.

The study and report for the French Government on a railway tunnel to cross the Seine near Le Havre to connect the State Railways on either bank.

The study and report for the Moscow-Kazan Railroad on the crossing by tunnel of the Volga River near Nijni-Novgorod.

The report on large reclamation projects for Bombay, India.

The reports on rapid transit development in Montreal, Que., and Toronto, Ont., Canada.

In his personal appearance, Mr. Jacobs was a man of striking presence and of a manner and poise most impressive. As an engineer he combined technical knowledge and good judgment, based on his long experience, with administrative and executive ability of the highest order. Many of his most important works were carried out without the medium of a contractor, with engineering and construction forces organized and directed by him for the owner. His courtesy and consideration to each of his numerous assistants and subordinates on the many projects he undertook during his busy life were never failing. Each of his staff felt that service under him was a valuable experience, and his magnetic personality inspired his men with loyalty and enthusiasm which bound them to him and to his work with bonds stronger than steel.

In his private life, Mr. Jacobs showed the same charming personality and magnetism. He made no enemies, but many close friends who were warmly and loyally attached to him. Writing of him, one of his old friends, a learned and honored Justice of the Supreme Court of New York, says:

"So many incidents of our long association come back to me and through all appear the cool, courageous, cheerful methods of our dead friend. He was at first a stranger in a strange land, but not for long. His ability, his knowledge of men, his courtesy and perseverance made their mark in the community. Tunnels under the rivers are common things nowadays but the pioneers had to struggle hard to make the financiers of twenty-five years ago appreciate the necessity and value of underground transit. * * * I am sorry he has gone. He made a great name for himself in his profession."

He had been a Member of the Institute of Mechanical Engineers since 1876 and a Member of the Institution of Civil Engineers since 1885. He was one of the early members of the Engineers' Club of New York, and had been a member of a number of clubs in New York City before he retired from active business there in 1916. In London, he maintained his membership in the City of London, the Royal Automobile, and the Royal Societies Clubs.

During the World War, Mr. Jacobs was very proud to have been associated with the production of ammunition and ordnance as a member of a Local Production Committee in the South Wales District.

Mr. Jacobs was an enthusiast in the development and use of motor cars. He was one of the early owners of a steam wagon and operated the various types as they appeared. In his religion he was a member of the Church of England. His home for many years before his death was "The Old Rectory House" at Wimbledon Park, near London. This was an ancient and beautiful Elizabethan house which he enlarged and modernized. During 1919, he had bought a summer home, a place called Glan-y-mor, at Laugharne in Caermarthenshire, South Wales,

which was ready for him at the beginning of September. He had only been in this house for a few days when he died as a result of cerebral hemorrhage. His last conscious thoughts were for those near and dear to him, and he left on his long journey with the same calm courage which had always been his. He was buried at St. Augustine's, in Penarth.

His loss leaves a gap which his friends and associates will not fill with his equal, and there is not one who knew him who will not remember him as an inspiring leader, a great engineer, and a thorough man.

Mr. Jacobs was elected a Member of the American Society of Civil Engineers on May 4th, 1904.

JULIUS LILIEN JACOBS, M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1918.

Julius Lilien Jacobs, son of Morris and Rachel Jacobs, was born in Atlanta, Tex., on April 6th, 1882. He received his early education in the public schools of Atlanta, and obtained a First-Grade Teacher's Certificate at the early age of 13 years. He entered the University of Texas at that age, and was graduated from there in 1899 with the degree of Bachelor of Science. He remained out of school one year, and entered Cornell University in 1900, from which he was graduated with Sigma Xi honors in 1904, with the degree of Civil Engineer.

In June, 1905, he went with O. M. Leland, M. Am. Soc. C. E., Professor of Geodesy and Astronomy at Cornell University, on the Alaskan Boundary Survey for the United States Government; he returned in September of the same year. Shortly after this he was employed by the Phœnix Bridge Company, as a Designer and Draftsman, with which company he remained about six months.

From September, 1905, to the early part of 1907, Mr. Jacobs was engaged as Engineer by the Chicago and North Western Railroad Company in connection with its new road between Pierre and Rapid City, N. Dak.

In 1907 he entered the St. Louis office of James Stewart and Company, Incorporated, of New York City, General Contractors, and remained with that firm until the time of his death. He was first employed as Civil Engineer on the Missouri, Kansas and Texas Railroad Terminals at Parsons, Kans., and later was sent to Fort Worth, Tax., to erect the freight shed of that railroad. Shortly after completing this work he was sent to New Orleans, La., in connection with work which the company was constructing in that vicinity.

In November, 1908, he went to Houston, Tex., to erect the First National Bank Building in that city. While in charge of this job, as Superintendent, he obtained additional work for his company, and was made Southern Manager, offices being opened in Houston. Mr. Jacobs handled a vast quantity of construction work while there, among which were piers, bulkheads, and warehouses at Galveston and Texas City, the Galvez Hotel at Galveston, Library and educational buildings at Austin, for the University of Texas, several buildings for the Rice Institute at Houston, and also many warehouses at Houston. It afforded him much pleasure to have erected the buildings for the University of Texas, his Alma Mater.

In January, 1916, Mr. Jacobs moved to Norfolk, Va., where he secured for his company the contract to build two large merchandise

* Memoir prepared by S. S. Jacobs, Assoc. M. Am. Soc. C. E.

piers and extensive bulkhead works for the Norfolk and Western Railroad Company in that city.

During the World War activities, Government work in the vicinity of Norfolk, amounting to about $12 000 000, was carried on under his direction. This included two large concrete shipways, a reinforced concrete warehouse, office building, and shop improvements for the Newport News Shipbuilding and Dry Dock Company at Newport News, Va. He also built four large apartments at Newport News, Va., as a Housing Project for the United States Shipping Board, Emergency Fleet Corporation, piers and warehouses for the United States Army Engineers Depot, at Norfolk, Va., the Ordnance Depot at Pig Point, Va., and piers and extensive bulkhead work at the Norfolk Naval Base, for the United States Navy. Shortly before his death he secured a contract for another large pier at the Naval Base, and work on this had just been started when he passed away.

On account of the urgency of this war work, he kept in close touch with it all, was a frequent visitor to the different jobs, and at all times was familiar with the detail work. His rule was to settle questions immediately; he would never allow matters to remain unsettled for any length of time.

Mr. Jacobs was an untiring and conscientious worker, and weighed all questions with justice to both his company and its clients. He was a man of high principles and strong convictions, and always stood up for what he thought was right.

His sterling character and sense of justice to his fellow-men won him many friends, and all his co-workers were eager to please him. He was kind and philanthropic, and was ever ready to give counsel and assistance to those who needed it. Much of his time was devoted to charitable organizations, in which he took an active interest.

He was a great reader and student, and kept in touch with current news and all large engineering projects. He made a close study of the latter, and possessed a remarkable faculty for remembering details. Although only 36 years of age, he had achieved wonderful success, and was recognized as a leader among men.

Mr. Jacobs was a member of the Texas Engineering Society and the Engineers Club of St. Louis. He was elected Junior of the American Society of Civil Engineers on September 1st, 1908, an Associate Member on February 28th, 1911, and a Member on March 2d, 1915.

TIEN YOW JEME, M. Am. Soc. C. E.*

DIED APRIL 24TH, 1919.

By the death of Tien Yow Jeme, at Hankow, China, on April 24th, 1919, China lost one of her most useful engineers in the construction and development of the railways of the country. He was a pioneer among Chinese engineers and, on account of his long service with the Chinese Government railways, his position was unique, and he was credited with enjoying the complete confidence of his Government. Practically his whole life was devoted to the construction of railways, and the story of his career nearly covers the history of railway development in China. He first became a National figure in 1909, when he completed the Peking-Kalgan Railway, a line 125 miles long, through difficult mountainous country, connecting the capital of China with the historical frontier mart of Mongolia. In completing this line, he was the first Chinese engineer to build a railway without foreign help in any capacity.

Tien Yow Jeme was born in Canton, China, on April 26th, 1861, so that he was 58 years old at the time of his death; by the Chinese calendar and the Chinese method of reckoning age, he was 59 years old. His grandparents came from the Province of Anhwei, having migrated to Canton where his father was attached to a mercantile firm.

His education was acquired principally in the United States, where he went in 1872, at the early age of 11. He had studied for a short time in the Chinese Educational Mission School, at Shanghai, previous to being sent to the United States with thirty other youths, comprising the first group of students to be educated there, at the expense of the Chinese Government. During the course of four or five years, the Chinese Government sent altogether 120 students to the United States, and of that number, Mr. Jeme was about the youngest, having been a mere child when he left Shanghai. He was sent directly to New Haven, Conn., and placed in charge of an American family for training, eventually entering the New Haven High School, where he prepared for college. He was among the first of his countrymen to pass college examinations and, in 1878, he was admitted to Sheffield Scientific School of Yale University. On account of his aptitude for the sciences and for mathematics, he took the course in Civil Engineering. He was a most brilliant student in mathematics, and captured the prize in that subject in both his Freshman and Junior years. He was graduated from Sheffield with a Ph. B. degree in Civil Engineering, in 1881, at the age of 20.

The original Director of the 120 Chinese students was Dr. Yung Wing, the first Chinese to have been graduated from Yale, and, in

* Memoir prepared by J. G. Wong, Assoc. M. Am. Soc. C. E.

later years, those students came to be known as "Yung Wing boys". Out of the number, one has been a Premier, one a Cabinet Minister, one an Ambassador, and numerous others have been Railway Directors and Customs Commissioners. On the whole, they have done well, and are worthy products of American education. Intrigues on the part of the Chinese Mandarin Hu caused Dr. Yung Wing to be transferred to be Secretary of the Chinese Legation at Washington, D. C., and Mr. Hu himself became the Director. He had no knowledge of English and was not in the least in sympathy with any new ideas. He had been sent out by the Manchus to watch the activities of Dr. Yung Wing who, the Court feared, might lead the youths astray. After having served as Director for some time, he sent a lengthy memorial to the Throne to the effect that the young men were fast being foreignized, and he thought that on their return they would contaminate China and introduce new and demoralizing ideas; he, therefore, recommended their immediate recall. The result of this action brought about the recall of the 120 students in the autumn of 1881. It was with great reluctance that they returned to China, as several of them were within a few months of receiving their university degrees. Of the 120 young men, only two went home with degrees from the University; they were Mr. Jeme and his classmate at Sheffield, Owyang King, at one time Chinese Consul at San Francisco, Cal.

On their arrival at Shanghai, the students were ordered by the Government to report to the Shanghai Commissioner of Foreign Affairs, who was to distribute them to various places for further training and reform, in order to fit them for Government service. They were literally looked on as plotters, and their every movement was secretly watched by Government agents. Mr. Jeme with fifteen others was sent to the Foochow Arsenal, on the coast opposite Formosa. While at the Arsenal, he took a course in navigation in the Naval School there, and studied under a foreigner by the name of Taylor. After one year of study, he received his certificate and was immediately placed on board a cruiser in the Chinese Navy for further training to become a naval cadet. After a short period on the cruiser he was asked to return to the Arsenal as a teacher in the Naval School. Soon afterward, war with France was declared, and the French squadron began its attack on the Arsenal, causing confusion in the school. Just at this time the Viceroy of Canton requisitioned Mr. Jeme to go to Canton to build fortifications. His most notable feat while at Canton was his complete survey of the coast line which was mapped out for the first time.

After peace with France was declared, Mr. Jeme was attached to the Canton Military and Naval Academy as an Instructor, where he remained until he was called north by Wu Ting Fang, the former Chinese Minister to the United States, to join the railway service at

Tientsin. It was the opportunity for which he had been waiting, the chance to make use of what he had acquired at Yale and to practice the profession to which he had dedicated his life. Wu Ting Fang was at that time the Director of the Tientsin Railway Company.

Mr. Jeme first joined the railway service at Tientsin in 1888, and, thereafter, for 31 years, he was continuously on the payroll of at least one railway. His first appointment was as Cadet Engineer, an assistant to a Resident Engineer. The Tientsin Railway Company was being financed by a British loan, consequently, the Chief Engineer, and all the principal engineers, were Englishmen. The Chief Engineer, Mr. C. W. Kinder, was in charge of the building of a line from Tientsin, passing the Tangshan coal mines, to Shanhaikwan, a Customs outpost on the Manchurian border. The ultimate object of the British was to extend this line past Shanhaikwan to the metropolitan city of Manchuria, as well as to build a line to Peking on the other side of Tientsin, thereby giving a through connection between Peking and Mukden *via* Tientsin, the greatest commercial city in North China. In the construction of the line between Tientsin and Mukden, Mr. Jeme took an active part on both sides of Shanhaikwan. By steady work, he was promoted successively to be Resident Engineer and District Engineer, and, invariably, he was given charge of the end section, the hardest, on account of being the farthest from the base, and, consequently, the most difficult to which to get material and supplies.

At first, the line between Tientsin and Peking was not allowed to be built on account of strong opposition in the capital. It was feared by the leading officials that, with a railway connecting Tientsin with Peking, the position of the capital would be greatly weakened; besides, there was bound to be local opposition from the natives when the railway people started to build through the many ancestral graves in the country districts. It was not until Li Hung Chang came to the rescue that work on the line was started, but the condition was imposed on the engineers that the railway must not come nearer than 10 miles of the capital. Therefore, the line was built to a small unimportant town named Fengtai, about 10 miles south of Peking. Some time after the completion of the railway from Tientsin to Fengtai, Prince Kung wished to go to Tientsin, and, in making the trip, he first had to travel by his slow-moving mule-cart from Peking to Fengtai, taking nearly a whole day to cover the 10 miles. He then boarded the luxurious train and covered the remaining 81 miles in about 3 hours. The difference in speed and comfort was too great to pass unnoticed, so when the Prince returned to Peking, he lost no time in removing the restrictions which had prevented the railway from approaching the capital. It was on the extension of this 10 miles that Mr. Jeme served as Resident Engineer. Later, when the line from Tientsin to Mukden was completed, both lines radiating from Tientsin were amalgamated into

what is known to-day as the Peking-Mukden Railway. In all, Mr. Jeme was connected with the Peking-Mukden Railway for 12 years, during which time he was always on active construction work.

During the Boxer trouble in 1900, when all the northern railways had suspended work, Mr. Jeme joined the Pinghsiang-Liling Railway, as a Construction Engineer. This line was being built to develop the coal mines at Pinghsiang, Kiangsi Province, and to bring the coal to a water route in Hunan Province. The Boxers had destroyed a part of the Tientsin-Peking Line, which caused the Allied troops to occupy the railway, as an English loan was involved. However, as soon as the rising was crushed, the railway was returned to China, and Mr. Jeme was asked to go back to the north to resume his former connection with the company.

In 1901, after the Boxer movement was quelled, the Imperial Court, on its return to Peking from Sianfu, Shensi Province, decided to go to the Imperial Tombs at Hsiling for ancestral worship. On September 20th, 1901, an Imperial Edict was issued expressing the desire of the Court, and, according to the Chinese custom, an Imperial road must be built. In this case, the suggestion of a railway was adopted, and the Court appointed Mr. Kinder as Chief Engineer to survey and construct the line. The French Minister at Peking at once presented a protest against Mr. Kinder's appointment, and asked for the appointment of a French engineer, claiming that, as the proposed line was to branch out from the Peking-Hankow Railway, a French loan line, French engineers should be entrusted with the work. Both the French and the English interests used every means in their power to get the appointment for an engineer of their own nationality, but as they could not compromise, they were forced to agree to the appointment of a Chinese engineer. Mr. Jeme was the logical choice, and, accordingly, he was made Chief Engineer of the Hsiling Railway.

The controversy between the French and English had consumed much time, and only about four months remained before the date set for the Imperial worship. The trains had to be ready by February 22d, 1902, for the local officials who had charge of making the preparations for the worship. Four months to survey, build, and run trains over 28 miles of virgin country! It was in the middle of winter, when the rivers were all frozen, making transportation of material troublesome; material was scarce and difficult to transport. Old rails were borrowed from the Peking-Mukden Railway, straightened, and used for sidings; the sleepers were spaced farther apart than for ordinary practice, and the large structures were of a temporary nature. During those four months, Mr. Jeme worked about 15 hours a day, and, for many nights, he could not sleep, because he knew that if he failed to have the trains running, he undoubtedly would have to forfeit his life. It was his first trial, and was accomplished under great handicaps, but,

fortunately, he was able to satisfy the Court, and the trains were operated for the Imperial Worship. When the Imperial Court had returned safely to Peking, he was rewarded by the rank of Prefect, a great honor in those days. He was also retained to make the roadbed a permanent one. From that time, his name began to be known throughout official ranks, and he became a great favorite of Yuan Shi Kai, then Viceroy at Tientsin, who had had principal charge of the arrangements for the Imperial Worship. Mr. Jeme never forgot the men who had helped him on this work and who had made his success possible.

Mr. Jeme's next important work was in connection with the taking over of the Tao-Ching Railway by the Chinese Government. This road had been built by the Peking Syndicate, an English corporation operating coal mines in North China. The railway is a branch of the Peking-Hankow Line and taps the great anthracite coal deposits of Shansi Province. Mr. Jeme represented the Chinese Government in the valuation of this line, and, later, in 1905, the Chinese Government paid the Peking Syndicate £614 600, the valuation set by Mr. Jeme in conjunction with the General Manager of the Syndicate.

Just about the time that the Hsiling Railway was completed with a permanent roadbed, Mr. Jeme was called to Canton on account of the death of his father. While at Canton, he was asked to join a commercial railway starting at Swatow on the coast. He subsequently went to Swatow, but only stayed there about two months, when he left for Shanghai, to accept a position as Consulting Engineer to the Shanghai-Nankin Railway Administration, which position he retained for nearly a year.

After the Allied Command had handed back the Peking-Mukden Railway to China, the line did a tremendous business and had plenty of money in its treasury, after deducting all operating expenses and meeting all loan obligations. Naturally, the Director wished to make use of this surplus for the construction of more lines; consequently, he recommended the building of the Peking-Kalgan line. Friends of Mr. Kinder in the capital at once started a campaign for his appointment as Chief Engineer of the proposed line. This seemed to be a logical choice, as the money for the proposed line had come from the surplus fund of an English loan railway, and, besides, Mr. Kinder was admitted to be a capable engineer of proven ability. However, there was great disappointment among his many admirers when it became known that the Russian Minister at Peking had handed the Chinese Government a protest against his possible appointment, pointing out that China had an understanding with Russia that any railway built beyond the Great Wall toward Mongolia must be financed by Russian capital and built by Russian talent. However, if the Chinese themselves could furnish their own money and men, he had no objection. By that time, Yuan Shi Kai had become a powerful man in the Cabinet, and he took up this challenge of the Russian Minister. He at once

sought audience with the Emperor to whom he urged the building of the Peking-Kalgan line; he knew just the man who could do the work, and he recommended that Mr. Jeme be appointed Chief Engineer. Yuan had implicit faith in Mr. Jeme's ability to carry out the project, and, later, it was shown that he had not misplaced his confidence. Thus, it came about that Mr. Jeme received the appointment over his former chief for the second time. It had not been of his own seeking, for he had great respect for Mr. Kinder. It was the inevitable, brought about by the peculiar conditions in each case, for Mr. Jeme was not a man to intrigue for another man's job. Besides appointing Mr. Jeme as Chief Engineer, Yuan Shi Kai had the foresight to make him Vice-Director concurrently, thereby giving him a voice in the Administrative Department. About two years later, he was promoted to be the Director. As the Chief Engineer, Mr. Jeme personally located the line and fixed the grades, and construction began immediately.

It was in the construction of the Peking-Kalgan Railway that Mr. Jeme gave proof of his greatness as an engineer and as an administrator. He inspired his men by his own hard work, and told them that they must succeed, as the eyes of the world were watching them. He impressed on them that whether they failed or succeeded, it was not their own failure or success, but must be shared by many generations to come. There was great team work throughout the ranks, for the men realized the full significance of their undertaking; and to those builders, their Chief, and the Chinese in general, the Peking-Kalgan Railway will stand forever as a monument to their ability and efficiency, and shows what the Chinese are capable of accomplishing when the right leader is found among them.

The Peking-Kalgan Railway cost about 7 000 000 taels complete. Mr. Jeme estimated the cost of the line at 7 223 900 taels, the work to be completed in four years. In his original estimate, he requisitioned for 1 800 000 taels per year, but was given only 1 300 000 taels during the first two years. In spite of this reduction, he completed the work within the scheduled time, and 4% under his original estimate in cost. The average cost per mile was 56 000 taels, equal to about $39 200 in gold, at the rate of exchange at the time. Whatever the critics may still have to say, the fact remains that a standard in construction cost has been set by a Chinese for the Chinese Government railways. There were reasons for this low cost. First, all the Chinese engineers dealt directly with the contractors, therefore, interpreters who might cause leakages in payments from the railway office to the contractors were not necessary. Then, there was no commission to be deducted at every purchase, as is the case for all foreign loan railways when the foreign purchasing agent is allowed the commission.

After the completion of the Peking-Kalgan Railway, in 1909, Mr. Jeme was engaged to carry on the work of extension. He retained his position on this railway until the end of 1911, when the Revolution

upset all departments of the Government service. At the opening ceremonies at Peking, Mr. Jeme received a great ovation from the people. It was one of the proudest moments of his life, equalled by only two other occasions, when he was praised by Dr. Yung Wing in the presence of all his schoolmates for capturing the Sheffield Freshman Prize in Mathematics, and his grand reception at Canton, when he went there to become President of the Canton Section of the Canton-Hankow Railway. In recognition of his great work, Mr. Jeme was given the Honorary Doctorate Degree by the Government. He was also made an Adviser to the Ministry of Education and Consulting Engineer to the Ministry of Communications. In 1910, he was appointed Chairman of the Board of Civil Service Examiners, for the examination of more than 700 men who had been educated in foreign countries.

Directly after the opening of the Peking-Kalgan Railway to traffic, the Viceroy of Szechuen Province sent a petition to the Throne for the second time, asking for the release of Mr. Jeme from Peking, in order that he might take up the construction of the Szechuen Section of the Hankow-Szechuen line. As everything was running smoothly on the Peking-Kalgan Railway, Yuan Shi Kai allowed Mr. Jeme to accept the offer, but still retained him in the service of the Peking-Kalgan line. Accordingly, Mr. Jeme went to Ichang in 1909 to accept the position of Chief Engineer of the Szechuen Railway Company, a privately owned company. The Szechuen line was to start from Ichang, Hupeh Province, and was to open up the Province of Szechuen, with ultimate connection with Hankow. Ichang is a treaty port on the Yangtze River, about 1 000 miles from Shanghai, and is situated just at the mouth of the famous Yangtze Gorges. The Railway Company had previously engaged a young Chinese as its Locating Engineer. He had already spent several years in the Szechuen hills, and Mr. Jeme's part on his arrival was to start the construction of the line. Mr. Jeme did not favor the located line which went inland, away from the Yangtze River, and was, therefore, much longer than a river line. He was of the opinion that a river route was better. However, as several years had already been spent in location work, the Company would not allow any more time for further investigations. The shareholders were clamoring for results; they wanted to see trains running. The natives throughout the Province were curious to know what a railway was like, they had heard so much of its wonderful performance and yet after spending so much money and waiting for so long, there was no sign of one. In later years, whenever a party of engineers went into Szechuen Province, the natives would invariably say, "coming again"? Mr. Jeme's opinion concerning the river line was afterward confirmed by surveys made during 1913 and 1914 by American engineers under Richard Wood Randolph, M. Am. Soc. C. E., Chief Engineer of the Szechuen Section of the Canton-Hankow-Szechuen Railways. In 1911, the Szechuen Railway Company was taken over by the Govern-

ment and this action was one of the chief causes of the Chinese Revolution. After the Revolution, the two trunk lines—Hankow to Canton, and Hankow to Szechuen—were amalgamated into one system to be financed by the so-called Group Bankers Loan, composed of British, French, German, and American capital, and the Szechuen line, now known as the American Section, was given over to the Americans as their field of activity.

In 1910, while he was at Ichang, busy getting construction work started, Mr. Jeme was informed that he had been elected President of the Canton Section of the Canton-Hankow line, a privately owned railway, even to this day. After construction at Ichang was under way, Mr. Jeme left for Canton to take up his newly added duties. In order to save money for the Company, he also acted as its Chief Engineer. He had been Consulting Engineer to the Lock-Tung Railway since 1909, so that during 1910 and the greater part of 1911, he was holding six different positions concurrently on four different railways. His time was spent between Peking, Ichang, and Canton.

When the Revolution broke out in 1911, Mr. Jeme was devoting his energies to the extension of the railway at Canton, and the Revolution gave him the chance to prove his faithfulness as its custodian. There was great excitement in and around Canton during the early period of the Revolution. Everybody who could afford it went to Hong Kong for protection under the British flag, but Mr. Jeme remained on the job. His staff threatened to desert him, and his many friends advised him not to take any chances by remaining in Canton. He called together the various chiefs of each department of the railway, and told them that he was going to stick to his post, but that if any one among them was afraid to stay, he could go; however, before they left, everything must be handed over to him or his representative in perfect order. In the end, nobody left, the trains were kept running throughout the Revolution, and the railway did not suffer in any way. A neighboring railway was looted after its Director and principal heads had deserted the place at the first sign of trouble. Everything that could be carried off was taken, and it was rumored that the remaining staff, mostly of the lower class, looted the railway themselves.

With the abdication of the Manchus, the country was somewhat restored to order, and the Canton-Hankow-Szechuen Railways System was organized. President Yuan Shi Kai sent a delegate to Canton to bring Mr. Jeme to the capital for appointment as Associate Director-General of the System. He was especially charged by Yuan to look after the technical part of the railway administration at Hankow. At the beginning the Director-General's post was reserved as a pension for some Revolutionary hero. It was passed from hand to hand during the first year, and, finally, a politician got the job, but only retained it for about six months when he died. Yuan then ap-

pointed Mr. Jeme as Director-General, the date of the appointment having been June, 1914, and from that time until his death, he retained this position. While he was Director-General, on the British Section, the portion between Wuchang and Changsha was completed and opened to traffic; on the German Section, a roadbed was completed for 164 km., but the tracks had been laid for only a short distance when the war stopped further work; on the American Section, no construction was attempted.

Besides the honors already mentioned, Mr. Jeme was appointed to represent China on the Allied Technical Board at Harbin. In 1916, he received the LL. D. degree from the British Colonial University at Hong Kong. He was Chairman of the Technical Committee in the Ministry of Communications, Vice-Chairman of the National Communications Conference, Honorable Member of the Board on Railway Laws and Regulations, Member of the National Transportation Committee, and he was twice decorated by the Chinese Republican Government. In 1916, on the thirty-fifth anniversary of his Class, his Alma Mater offered him the honorary degree of M. A., but he was unable to go to the United States to receive it.

During the construction of the large bridge across the Yellow River, on the Tientsin-Pukow Railway, the work was held up by the local gentry who demanded the addition of more spans. The natives drove off the German engineers and the German contractors and refused to let them proceed with the work. The Director was in despair, as he was unable to cope with the situation. He asked for help from the Government, and Mr. Jeme was sent to investigate and commissioned to do what he could to settle the matter. He not only explained everything satisfactorily to the gentry, but also secured their consent to quarry rubble from a near-by hill, thereby cheapening the cost of rubble. In this mission which required tact and diplomacy, he saved the Government many thousands of dollars.

He was the first to introduce the use of rack-a-rock in China during the construction of the Peking-Kalgan Railway. On every railway of which he had charge, he set a high standard in pay for the Chinese engineers. He believed firmly that Chinese engineers should receive the same pay as European engineers of the same rank and for the same work. He had a warm place in his heart for young engineers, especially for those who had been educated abroad. To them he was an inspiration, and was always looked to for guidance. He was always accommodating to any one who sought his help.

The following story is a concrete example of his silent fight for the good of Chinese engineers. While he was still Associate Director-General, the German Chief Engineer, Mr. Linow, sent an official communication to the Director-General for sanction to send for a number of German engineers of all ranks to replace the Chinese engineers.

Mr. Linow said in his letter that "the Chinese engineers were no good" Mr. Jeme was aroused at once by this letter and strongly advised the Director-General that under no consideration must such an application be sanctioned. He clearly saw that if this was allowed, it would create a precedent for the wholesale importation of foreign engineers and likely to be followed by other foreign loan railways. In time, Chinese engineers would disappear from the railway service. The Director-General not wishing to get himself into trouble with the Germans, wired from Peking that the Associate Director-General would have the power to decide the question. Accordingly, Mr. Linow was asked to come to the Director-General's office for consultation. On his arrival, he was asked to re-read his letter to see if it was correct. Mr. Linow replied that the letter was quite correct. Then Mr. Jeme asked him what he meant by the statement, "Chinese engineers were no good". Mr. Linow replied that he only meant those whom he had who were not up to the standard. Whereupon, Mr. Jeme gave the letter back to Mr. Linow and told him that the statement was too broad, he must alter that letter. If, however, Mr. Linow thought the Chinese engineers whom he had were not up to the standard, better ones would be found until he was satisfied. Mr. Jeme also told him that if any Chinese engineers were dismissed, the vacancy must be filled by Chinese according to the loan agreements. He also expressed the opinion that he did not believe it would be just for any foreign loan railway to import foreign engineers to fill all the ranks; it was all right for experienced foreign engineers to take the principal positions, but the lower ranks ought to be reserved for the Chinese to get the practice and training. In the end, Mr. Linow saw the justice of Mr. Jeme's refusal, and acted accordingly. This was one of many such cases, where Mr. Jeme took the burden on his own shoulders for the good of the Chinese in general.

As time goes on, Mr. Jeme's record of achievements will undoubtedly receive greater recognition. He was always a clear thinker and a man of few words. In his long years of toil, he was never known to have made an error in judgment, and although a man of strong convictions, he was always open to argument and ready to receive suggestions and give them consideration. He was never known to fail in keeping a promise, and was always punctual at his office, at meetings, or at social functions. Between him and those who were with him through his early struggles, there existed a deep and sincere affection. To the older class of Chinese engineers, he was always their leader, one to whom they always went for advice and help. He was frank, open in his dealings, honest to the core, and above petty personal interests. He was a true gentleman at all times, courteous to all, and possessed of high moral conceptions.

He was a man of the most simple tastes, modest in the extreme, and never harbored resentments or belittled those who opposed him; he was never jealous of the man above him. He always considered himself as an engineer, and never mixed in politics. When the Revolutionary party asked him to join their ranks, he refused politely saying that he was only an engineer, one who was to make progress by his creative and constructive endeavors. He was one of the founders of the Chinese Institute of Engineers, and obtained its recognition by President Li Yuan Hung. It was his aim to give the Society a standing that would qualify its members to accept any technical post under the Government. Through his efforts, several thousand dollars have been collected toward a building fund for the Society, and Mr. Jeme secured temporary quarters in Peking for it. Whenever he was in Peking, he always stayed at thē Society headquarters in preference to the luxurious hotels.

He was a lover of the American game of baseball and, on his return trip in 1881, he played on the All-Chinese team organized by the returning students. The All-Chinese team played an exhibition game with an Oakland semi-professional team and surprised the Americans by their excellent playing. He took a keen interest in the Yale ball team and followed its history through his alumni papers. He was enthusiastic over tennis, and occasionally played a set or two with his children. He lived a clean life, and had never known sickness until about a year ago, when he was first attacked with dysentery from which attack he never fully recovered. During the past year, his health began to fail rapidly, but he stayed at his work, hoping all the time that he would soon be well again. He was in poor physical condition when President Hsü sent him to Harbin to represent China on the Allied Technical Board. The long journey and the strain of the work had their telling effect, for three days after his return to Hankow on a short leave, he was advised to enter the hospital for careful treatment.

On his arrival at Hankow, he had decided to request President Hsü to release him from all public duties, and had asked his Secretary to prepare his resignation, which he had just time to initial, when he passed away at the hospital. According to the doctor who attended him, his death was due to heart failure and overwork.

In his valedictory petition to the President, Mr. Jeme urged the training of more engineers, with every chance for practice, recommended the appointment of a competent expert to succeed himself on the Allied Technical Board, and, finally, the speedy completion of the Canton-Hankow-Szechuen Railways, especially the Szechuen line, on the great future of which all engineers, travelers, writers, merchants, and missionaries have agreed.

On his death, President Hsü issued a mandate in which he commanded the National Historian Committee to record Mr. Jeme's deeds in the history of the Nation, donated the sum of $2 000 toward defraying the funeral expenses, and ordered the Civil Governor of Hupeh Province to represent the Government at the funeral services at Hankow in offering sacrifices. Mr. Jeme's many friends at Canton, Shanghai, Tientsin, Hankow, and Peking held memorial services to pay him tribute.

Mr. Jeme was elected a member of the Institution of Civil Engineers of Great Britain in 1894. He was also a member of the Royal Academy of Arts and of the Shanghai Engineering Society (Europeans), President of the Chinese Institute of Engineers, President of the Hankow Foreign Educated Men's Club, member of the Chinese Railway Association, and while at college, a member of the Alpha Phi Fraternity.

He was married in 1887, at Canton, and his widow, five sons, two daughters, one brother, and one sister survive him. His two eldest sons had only returned from the United States, in February, 1919, one having been graduated from Yale in Civil Engineering and the other from Tri-State College in Mechanical Engineering.

The following telegram and letters show in what high esteem Mr. Jeme was held by those who knew him and who had been associated with him:

The following telegram of condolence was received from the Hon. Paul S. Reinsch, the American Minister to China:

"We are greatly shocked by information of death of Dr. Jeme whose efficient services to Chinese railway development and whose eminent qualities of character gave him a distinguished position of confidence with all Americans."

John F. Stevens, M. Am. Soc. C. E., President of the Inter-Allied Technical Board, wrote as follows:

"Dr. Jeme was known to me somewhat by reputation before I had the pleasure of his acquaintance, and I am glad to believe, his friendship. During the short time I was associated with him in our work on the Allied Technical Board, I was deeply impressed by his quick grasp of any matter which we had under consideration, his rapid, and to my mind, excellent judgment.

"While a man of few words, he was decisive, and we all greatly respected his opinions, and one could readily see that, coupled with a thorough technical knowledge of railway problems, he possessed what was even more valuable, strong common sense and a thorough belief in his convictions.

"Above all, he was a gentleman, and the great pleasure I have in the memory of his association, is only equalled by the sense of loss not only personal to all of us, but a very great one to his country."

C. J. Carroll, M. Am. Soc. C. E., Chief Engineer of the American Section of the Canton-Hankow-Szechuen Railways, wrote:

"During the past two years that I have been associated with the late Dr. Jeme in affairs of the Szechuen-Hankow Railway, I have always found him deeply and personally interested in the ultimate success and completion of not only this railway, but also in a broader sense the comprehensive railway development of China.

"Dr. Jeme was recognized as the leading Civil Engineer of China on account of his technical ability, of which I had many occasions to observe, as a man of real accomplishment such as the carrying through to completion and success the Peking-Kalgan Railway, and as a man of broad ideals, interested and enthusiastic in the development of China's transportation system as a whole.

"All who came in contact with Dr. Jeme could not help but admire not only his ability as an administrator, a conceptor, and an engineer, but his interesting personality and gentlemanly instincts which inspired confidence.

"At the time of his death he was looking forward to the success of his great ideal, the opening up of Szechuen by a railway through the noted and difficult Yantze Gorges. By his death, China has suffered a great loss and I trust that further honors will be conferred upon this noted man by the nation."

The following letter was received from H. A. Raider, Assoc. M. Am. Soc. C. E., former District Engineer of the American Section of the Canton-Hankow-Szechuen Railways:

"The writer had the happy experience of having been associated professionally with Dr. Jeme Tien Yow, and expresses his deep feelings at the loss of a friend and of an engineer who had become of inestimable value to his country in the development of her numerous railway systems.

"Dr. Jeme was without doubt the ablest engineer that China has produced, and might well be called the Father of Engineering in China, because not only of having been the first Chinese Engineer to build and operate entirely with Chinese a large and difficult railway undertaking, but because he was always the first to contribute to, encourage, and maintain any society or measure that would promote the science of engineering in China.

"He was a man of great ability, possessing fine personal qualities, and commanding the good will and confidence of all his associates. He undertook with enthusiasm and characteristic attitude of duty, every assignment made him by his Government, even to his last, which was indirectly responsible for his early death.

"I trust the young engineers of China will set him up as a model, a man who followed his profession because his country needed engineers, when with his ability he might have amassed wealth in commercial life."

Mr. Jeme was elected a Member of the American Society of Civil Engineers on November 30th, 1909.

CASSIUS WILLIAM KELLY, M. Am. Soc. C. E.*

DIED DECEMBER 3D, 1918.

Cassius William Kelly, son of John Kelly and Sarah (Sigler) Kelly, was born on May 10th, 1848, in Pleasantville, Pa., where he received his early education. Later, he went to Yale College, and, unaided, worked his way through the Academic Department, and then through the Civil Engineering Course of the Sheffield Scientific School, receiving the degree of Bachelor of Arts in 1870, with a Phi Beta Kappa standing. He received the degree of Bachelor of Philosophy in 1872.

In June, 1872, Mr. Kelly entered the Engineering Department of the City of New Haven, and was connected with that department, in various capacities, for the remainder of his life—a period of forty-six years.

On January 18th, 1893, he was made City Engineer by the unanimous vote of the Board of Public Works, and served in that capacity until November, 1912, when he was made Consulting Engineer—a position for which he was eminently qualified, not only by his engineering ability, but also by his thorough knowledge of the engineering features and development of the city, and by his familiarity with the intricacies of charter provisions, contract formalities, and the legal technicalities affecting a city's structural growth.

Mr. Kelly's earlier work included a complete trigonometrical survey of the city; determining and establishing street lines and grades; complete charge of paving operations; and the charge of sewer construction during the year preceding his appointment as City Engineer.

In the design of deep masonry foundations for movable bridges he was conservative, and at the same time showed great originality in dealing with the problems involved. His designs for arch bridges of reinforced concrete were well worked out, and the finished bridges are not only adequate but also very satisfactory to the eye.

Mr. Kelly's remarkable memory, keen reasoning powers, and the acknowledged accuracy of his judgment rendered his decisions final and conclusive on many a disputed point.

In his profession, as well as in social intercourse, his absolute integrity and unvarying kindliness gained for him the love and high esteem of all who knew him.

Mr. Kelly is survived by his widow, formerly Frances E. Hart, and two daughters, Miss Miriam F. Kelly and Elsie L., wife of Mr. Grey W. Curtiss.

He was a member, and the Clerk, of the Dwight Place Congregational Church, a member of the New Haven Chamber of Commerce, and the Connecticut Society of Civil Engineers.

* Memoir prepared by Henry H. Gladding, M. Am. Soc. C. E.

Politically, he was a Republican, on general principles, but did not hesitate to exercise an independent discriminating judgment in local issues.

Mr. Kelly was elected a Junior of the American Society of Civil Engineers on March 1st, 1882, and a Member on June 6th, 1888.

PAUL SOURIN KING, M. Am. Soc. C. E.*

DIED OCTOBER 30TH, 1919.

Paul Sourin King was born at Philadelphia, Pa., on March 16th, 1845. Until he was fourteen years old, he was seriously hampered by the fact that he was crippled in both legs. As a boy he attended the public schools and, later, went to a small college in Westmoreland County, near Greensburg, Pa., but he was largely self-educated.

At the age of seventeen, Mr. King was employed by Herman Fisher, a Mining Engineer, at Ashland, Schuykill County, Pa., with whom he stayed about one year. In 1865, he was engaged as Rodman with the Locating Corps of the Mahonoy and Broad Mountain Division of the Philadelphia and Reading Railroad, under Charles E. Byers, Chief Engineer, principally on work connected with the Port Clinton, Hamburg, and Kutztown Branch, and other branches of the road, until 1870.

From 1871 to 1875, Mr. King served as Chief Engineer of the Roanoke Valley Railroad Company, on the construction of 53 miles of that line in Virginia, his headquarters during this time being at Richmond, Va.

In 1875 and 1876, he was engaged as Engineer in charge of the Virginia and Gold Hill Water-Works Extension, on the location and construction of a tunnel through the Sierra Nevada Mountains, as well as a pipe line, dams, etc., in the State of Nevada, and from 1877 to 1879, he was employed as Chief Engineer of the Union Island Reclamation Company on the location and construction of 30 miles of levee work, piling, damming sloughs, etc., in California.

In 1880, Mr. King went to Mexico as Assistant Engineer on the location of the Mountain Division of the Mexican National Railroad, and remained there until 1881. On his return to the United States, he was engaged with Messrs. Oliver Barnes and Charles McFadden, who, at that time, were grading about 30 miles of road in Elk, McClain, and Clearfield Counties in Pennsylvania. He was also employed as Division Engineer on the Pittsburgh, Youngstown, and Chicago Railroad, on the location of the Ohio River Bridge crossing, etc., in Pennsylvania.

About this time the construction of the South Pennsylvania Railroad, from Harrisburg to Pittsburgh, was under way, and Mr. King was appointed Division Engineer on the location of the road through the Blue Mountain Section in Pennsylvania.

Mr. Robert H. Sears, Chief Engineer of the Lehigh Valley Railroad, recommended Mr. King to the President of the Bethlehem Coal and Iron Company, and, in 1883, he was sent by the latter Company to Cuba as Chief Engineer of the Juragua Iron Company, Limited. Rail-

* Memoir prepared from information furnished by Henry King, Esq., and on file at the Headquarters of the Society.

road, on the location and reconstruction of 18 miles of difficult mountain road, as well as bridges and a pier for loading iron ore in the harbor of Santiago, Cuba.

On his return from Cuba in 1885, Mr. King was engaged by the Lehigh Valley Railroad Company to build a branch of that road between Buffalo and Geneva, N. Y., of which work he was Chief Engineer.

On the completion of this work, he established an office for the private practice of engineering as Consulting Engineer in New York City, which he maintained for more than 15 years. During this period, he went to South America with the editor of *Engineering News* and, in addition, traveled extensively, attending practically all the Worlds' Fairs.

Although, as has been stated, Mr. King was crippled in the early years of his life, he had recovered entirely from this condition; in fact, he frequently stated that he could outwalk any man in his party. He had never married.

To his friends Mr. King was known as a fine, noble, unselfish character—a man of scrupulous integrity and a high sense of honor—and it was this fact which brought to him important positions of responsibility and trust in engineering work.

Mr. King was elected a Member of the American Society of Civil Engineers on July 3d. 1889.

HERMANN LAUB, M. Am. Soc. C. E.*

DIED DECEMBER 10TH, 1918.

Hermann Laub, of Swiss ancestry, was born in Durmentingen, Wurtemberg, on April 14th, 1855, his family having moved there from Switzerland some years before that time.

At the age of twelve he entered the Preparatory School at Riedlingen, where, in addition to his mathematical studies, he took up English, French, and Spanish before entering the Higher Preparatory School at Biberach.

After pursuing the preparatory course at the Polytechnical School of Stuttgart for Civil Engineering, he was admitted as a full student, and was graduated as a Civil Engineer in 1879. He also pursued the course of civil engineering at the Munich Technical High School.

In 1880 Mr. Laub came to the United States, where he took a position in Denver with the Denver and Rio Grande Railroad. From 1881 to 1884 he was engaged in railroad location with the Mexican National and the Mexican Central Railroads. While connected with the Mexican National Railroad, he located a road from Salvatierra to Celaya, San Miquel, and San Luis Potosi. As Locating Engineer of the Mexican Central Railroad, he located a road from Tepic to Navaretta and San Blas, on the Pacific Coast, over some of the roughest country in Mexico. During his three years in Mexico, he located, in all, more than 500 miles of railroad.

At the conclusion of his work in that country, Mr. Laub went to Pittsburgh, Pa., where he accepted a position as Designer with the Schultz Bridge Company, in whose employ he remained from 1884 to 1887. Among the chief designs he made while with this company were those for the Junction Railway Bridge over the Allegheny River at 33d Street, Pittsburgh, the Indianapolis train-shed, and numerous other structures. In 1887 and until 1890 he associated himself with Gustav Lindenthal, M. Am. Soc. C. E., during which time the Smithfield Street Bridge and extensions were executed, and a bridge over the Monongahela River at Duquesne was built.

In 1890, Mr. Laub established himself in Pittsburgh as a Civil and Consulting Engineer, and from that time until his demise occupied and enjoyed a high place in the Engineering Profession, having always the esteem and high regard of his clients.

In 1908 he was engaged on the reconstruction of the Homeopathic Hospital, at Pittsburgh, Pa.; and in 1910 he was called upon to repair, at great risk, the 22d Street Bridge, without stopping traffic for a single day.

* Memoir prepared by A. L. Schultz, M. Am. Soc. C. E

For many years Mr. Laub was selected by the State of Pennsylvania as Bridge Engineer for the erection of numerous bridges throughout the State. He represented more than twenty bridge companies as expert Valuation Engineer during the period from 1909 to 1916 in condemnation proceedings for the sale of bridges to Allegheny County.

Mr. Laub's versatility enabled him to design various structures for special purposes, covering almost every form of industrial plant, in addition to long-span bridges of every design over the Monongahela, Allegheny, and Ohio Rivers, such as the McKeesport and Port Vue Bridge in 1909, the Brownsville Bridge in 1913-14, the Parkersburg, W. Va., Bridge, in 1915-16, and the Norfolk Bridge in 1917-18, also, in 1914-17, the repair of the Hot Metal Bridge for the Monongahela Connecting Railroad over the Monongahela River at Pittsburgh.

Among the chief industrial plants constructed under his supervision were the Chambers-McKee Glass Plant, at Arnold, Pa.; the McKee Glass Plant, Jeanette, Pa.; the Oliver Iron and Steel Company (various mills); the galvanizing plant for the National Tube Company, the Rochester Tumbler Company, and the Federal Glass Company, at Cambridge, Ohio.

Mr. Laub was one of those generally robust, earnest, and untiring workers whose keen judgment was always gladly accepted. His general disposition showed a marked reserve, but to his intimates he displayed a lovable attitude. Mr. Laub was always in good health, so that his sudden demise from heart failure on December 10th, 1918, came as a shock to all with whom he was acquainted.

On November 27th, 1884, he was married to Miss Rosa Hauber, at Jersey City, N. J., and she, with two sons and two daughters, survives him. One son was in the service of the United States Navy at the time of his father's death.

Mr. Laub was elected a Member of the American Society of Civil Engineers on September 1st, 1897.

GEORGE STEVENS LEAVENWORTH, M. Am. Soc. C. E.*

DIED AUGUST 8TH, 1919.

George Stevens Leavenworth was born on August 17th, 1858, at Charlotte, Vt. He was graduated from the University of Vermont in 1885, with the degree of Civil Engineer.

After serving a short time with the United States Coast and Geodetic Survey, and with the City Engineer of Burlington, Vt., Mr. Leavenworth, in March, 1886, took up location and construction work on the Burlington and Missouri Railroad, the Marietta and North Georgia Railway, the Knoxville Southern Railway, and the Missouri, Kansas, and Eastern Railway.

In April, 1896, he went into street railway and hydro-electric development work, on which he was employed, chiefly in Massachusetts and New York State, until about 1901, when he returned to steam railroad construction on the Manitoulin and North Shore Railway, the Algoma Central Railway, and the extension of the Little Kanawha Railroad.

In 1903, Mr. Leavenworth resumed street and interurban railway work as Chief Engineer of the Powers and Mansfield Company, of Troy, N. Y., in which work he continued until about 1912, when he engaged in railroad and mining engineering in the South. In this connection, he was employed with the Carolina and Yadkin River Railroad and the Tennessee Copper Company, continuing in that work until his death which occurred at his home in Ducktown, Tenn., on August 8th, 1919. He is survived by his wife, who was Miss Sarah H. Smith, of Charlotte, Vt.

Mr. Leavenworth was quiet and unassuming in manner and had a lively sense of humor. He had a keen analytical mind, and his aptitude for railroad location was very great. His integrity was never questioned, and his honesty of purpose impressed all who had dealings with him. He represented fully the highest type of engineer, and all who knew him have suffered a loss by his death.

Mr. Leavenworth was elected an Associate Member of the American Society of Civil Engineers on June 3d, 1903, and a Member on February 5th, 1907.

* Memoir prepared by W. H. Mansfield, M. Am. Soc. C. E.

MAXYMILIAN LEWINSON, M. Am. Soc. C. E.*

DIED SEPTEMBER 19TH, 1915.

Maxymilian Lewinson was born in Plock, Poland, on March 14th, 1855. After he was graduated from the Polytechnicum, in Zurich, Switzerland, in 1877, where he met Professor Ruleaux and others famed for their research work in various departments of science, he entered for a short time the employ of the German State Government.

About 1880, Mr. Lewinson came to New York City and met the trials of a stranger in a strange country with courage and determination, his education being his principal asset.

After a time, he became associated as Engineer with the late Leopold Eidlitz, an architect of great ability and extensive practice. In this capacity Mr. Lewinson was called on to solve many complex problems of construction, such as the rebuilding of the Assembly Chamber roof, at Albany, N. Y., and the reconstruction of Cooper Institute.

Mr. Eidlitz, although practicing architecture, was himself well grounded in the fundamentals of engineering, which fact created the tie leading to a close friendship with Mr. Lewinson that continued to the time of Mr. Eidlitz's death.

Later, Mr. Lewinson practiced his profession on his own account, and, still later, he formed a partnership under the firm name of Baker and Lewinson, Architects and Engineers.

Under the administration of Albert F. D'Oench, Architect, then Superintendent of the Department of Buildings, he entered the employ of the City of New York. Up to this time (1888), so-called practical builders seemed to have met the needs of this Department, and the educated engineer had failed to find recognition. However, the wonderful development in building operations and the dawn of skeleton building construction in the City of New York created a demand for an engineer of ability, and Mr. Lewinson found himself, therefore, the first engineer of this important Department of the City's activities.

Toward the close of 1889 he severed his connection with the Department of Buildings, and entered business as an Engineer and Contractor for iron and steel work, especially as related to buildings.

In March, 1891, he formed a partnership with the writer to engage in general engineering practice and as contractors for structural work. This affiliation proved to be most happy, and the firm enjoyed a prosperous career until the dissolution of the partnership in 1903. Mr. Lewinson then organized the corporation of Lewinson and Company, Engineers and Contractors, and became its President, which position he retained until his untimely decease.

* Memoir prepared by the late George A. Just, M. Am. Soc. C. E.

Mr. Lewinson was a contributor to the *Transactions* of the Society, and often took part in the discussion of papers presented at its meetings.

He became a member of the Engineers Club in 1890. He was also a member of the Museum of Natural History, a Director of the Manhattan Square Apartment Association, and for a term was Highway Commissioner of the Town of Scarsdale, Westchester Co., N. Y. The latter position he accepted as a public duty, giving freely of his experience and advice for the benefit of his fellow-townsmen, he having become a resident of Scarsdale.

Mr. Lewinson was married, but left no issue. An elder brother, Simeon Lewinson, engaged in mining operations in the West, survives him, as well as relatives residing in Europe.

Mr. Lewinson was a most lovable character. He delighted to entertain his friends, and always proved an admirable host. He was generous to a fault. To any one who applied to him for assistance it was promptly and cheerfully granted. He was a well-read man, with a broad general education, and this permitted him to discuss with intelligence and force, not only important questions in engineering, but also world politics, literature, and art. In his death, his professional and social acquaintances suffered a great loss.

Mr. Lewinson was elected a Member of the American Society of Civil Engineers on February 7th, 1894.

ROBERT EMMET McMATH, M. Am. Soc. C. E.*

Died May 31st, 1918.

Robert Emmet McMath was born in Varick, Seneca County, N. Y., on April 28th, 1833. One of the vivid recollections of his childhood was going with his parents to Little Falls, N. Y., to see the first passenger train go through that place on its trip from Schenectady to Utica.

When he entered Williams College from which he was graduated in 1857, he found that he was handicapped by the deficiency of his preparatory course, particularly in the languages, as compared with men who had prepared at Andover and other Eastern schools. While at Williams, he was a fellow student of the late President James A. Garfield, through whose influence he became a member of what was known there as the Anti-Secret Society.

In the latter part of October, 1858, Mr. McMath went West and settled in St. Louis, Mo. He entered directly on the active practice of his profession, his first work having been the design and reconstruction of a levee around Columbia Bottom at the junction of the Mississippi and Missouri Rivers.

From 1859 to 1862, he served as Deputy County Surveyor of St. Louis County, Missouri. In 1862, he entered the employ of the Government as an Assistant Engineer with the United States Coast Survey, under the late Alexander Dallas Bache, Hon. M. Am. Soc. C. E., as Superintendent. While awaiting orders, Mr. McMath made a reconnaissance survey of the environs of Philadelphia, Pa., until the spring of 1864, when he was ordered to Washington, D. C., to report to Gen. Meigs, Quartermaster Corps, U. S. A., for duty. Under orders from Gen. Meigs, he made a topographical survey of Arlington Heights. This property was owned by the Lee family, but had been confiscated by the Government for use as a burial ground for soldiers dying at or near Washington, and is now known as the Arlington National Cemetery.

Early in February, 1865, Mr. McMath, with others of the Coast Survey, was ordered to Nicaragua, where the Central American Transit Company, under grant from the Nicaraguan Government, was about to make a survey for the Nicaragua Canal. On this detail, he was engaged on a survey of Greytown Harbor and about 1 000 miles of the San Juan River which at that time was bordered on each side by an unpenetrable wilderness. Until their return to New York, the members of the party had not heard of the surrender of the Confederate Army or the assassination of President Lincoln, and during the voyage

* Memoir prepared from information on file at the Headquarters of the Society.

home had been continually apprehensive of capture by a Confederate cruiser.

In the fall of 1865, he received and accepted an offer of a position as Assistant Engineer with the U. S. Engineer Corps. His first work in this capacity was on the survey of the Illinois River from La Salle, Ill., to the Mississippi River. In 1867 and 1868, he had charge of a hydrographic survey of the Illinois River, and, in 1869, he was engaged as Assistant Engineer on a survey of the Arkansas River until September, when he was placed in local charge of the improvement of the Illinois River, as part of a canal and slack-water navigation from Chicago, Ill., to the Mississippi River. It was while he was engaged on building dams in the Illinois River that Mr. McMath applied the practice of lumbermen of using brush and stakes, with such ballast as might be available, a method afterward used to a great extent in similar work on the Mississippi and other rivers.

Mr. McMath remained on the work of the improvement of the Illinois River, until July, 1872, when appropriations having been made by Congress for the improvement of the Mississippi River, he was assigned, in August, 1872, to take charge of the improvements of the harbors of St. Louis, Mo., and Alton, Ill. In April, 1873, he was appointed Principal Civil Assistant on the improvement of the Mississippi River, between the Illinois and Ohio Rivers, and, in October, 1878, he was put in charge of special physical investigations thereof. He served with the Mississippi River Commission on similar work from 1880 to 1883, when he was appointed Sewer Commissioner of St. Louis, under Mayor Ewing, which position he held until 1891.

Mr. McMath had never had any experience with sewer work prior to his appointment as Sewer Commissioner, but when the opportunity of doing important and necessary work for the city thus presented itself, he did not hesitate to accept it. During his incumbency of the office, he originated the drainage scheme for what is known as the Vandeventer District of St. Louis, which had presented problems that had been stumbling blocks of long standing to the Department. Under his direction, a complete topographical survey of the city was made, which is still referred to in planning sewer extension. He also made a special study of the question of the size of sewers required to serve large areas, and embodied the results of these studies in a paper* which was presented before the Society. The formula given in that paper for determining the expected discharge of storm water from areas exceeding 100 acres, known to engineers as the "McMath formula", was widely accepted and used by engineers throughout the country. It has been stated that, in this formula, Mr. McMath "had no idea that

* "Determination of the Size of Sewers", *Transactions*, Am. Soc. C. E., Vol. XVI (1887), p. 179.

he was contributing anything but a method to engineering science and really did not think that he did."

From 1891 to 1893, Mr. McMath was engaged in the private practice of engineering as a Consulting Engineer, in St. Louis. In April, 1893, he was nominated and elected to the office of President of the Board of Public Improvements of St. Louis, and served in that capacity until 1901. In this position he was obliged to familiarize himself with all kinds of municipal problems. He was a prominent member of the City Hall Commission, under the control of which the present City Hall was built, and this building was constructed largely along the lines suggested by Mr. McMath.

After severing his connection with the City Government in 1901, Mr. McMath engaged again in the private practice of his profession as a Consulting Engineer, retaining his office in St. Louis, until he retired a few years before his death, on May 31st, 1918, to his home in Webster Groves, Mo.

Mr. McMath was married in 1859 to Miss Frances Brodie, of Detroit, Mich.

He was a prominent member of the Engineers Club of St. Louis and had been identified with its activities almost from its organization. He served as its Vice-President in 1885, and was elected its President in 1886. He also served as Club Librarian from 1891 to 1893, and was elected an Honorary Member in 1910. The following extracts are taken from a memoir of him, prepared by Robert Moore and John A. Ockerson, Past-Presidents, Am. Soc. C. E., and M. L. Holman, M. Am. Soc. C. E., for the Club:

"While his career was marked by unusual service, this was only natural for one of his remarkable ability. His integrity was as pronounced as his professional capabilities and considered both as a man and as an engineer nowhere in contemporary history can there be found a character more worthy of emulation. Sincere, always most assiduous in his effort to reach the best solution for the problems that he was set to master, a wide reader and a ready writer, his thoroughness gave his contributions to the affairs that claimed his attention the stamp of authority and finality. As one of his associates has aptly said 'when Robert McMath gave his opinion on a subject you knew that it was the result of careful, conscientious study, enhanced by a wide experience and determined by a splendidly trained mind.'

"During the period of his association with the Mississippi River Commission, Mr. McMath was regarded as the best informed engineer on river hydraulics in the country. The elaborate reports which he compiled from time to time were considered as standard and conclusive on many of the problems pertaining to the regulation of the river."

* * * * * * *

"Personally, though a man of few words, his friendship could always be counted upon, and he was generous to the last degree, both in

the assistance he rendered his colleagues and in the aid he extended to the young men associated with him who were just growing up in the profession.

"The life of Robert McMath presents one of those rare instances where personality has embraced intellect, sensibility and will of the highest order, each happily balanced against the other and forming a character of charm and power.

"Those of us whose privilege it was to have known him cherish the memory of our association together. The State and society have been enriched by his example of uprightness and his constructive achievements."

The following paragraph is quoted from Mr. McMath's autobiography, and is a fitting close to an account of his life:

"As is the usual lot of men, I have had opportunities for usefulness, and if it has not fallen to my lot to accomplish any great work, I have lived at a time when great advances were being made in matters affecting the welfare of men of all classes and kinds. The material advances and changes have affected all classes and conditions of mankind, and the most that any of us can claim is that we have not failed to meet the responsibilities of the hour and to do what was given us as our work."

Mr. McMath was elected a Member of the American Society of Civil Engineers on March 3d, 1880. He also served as a Director of the Society in 1889.

EDGAR MARBURG, M. Am. Soc. C. E.*

DIED JUNE 27TH, 1918.

Edgar Marburg was born on March 4th, 1864, at Hamburg, Germany, where his father had sent his family from New Orleans during the Civil War. His father, Herman Edward Marburg, was a man of artistic tastes and a lover of arts, who had planned to devote himself to languages but who finally engaged in business and succeeded so well that he became a wholesale merchant. He came to America in 1847, representing a Hamburg firm, and became a naturalized citizen in 1853. His mother, Clara Juergesen Marburg, was also a native of Germany. Edgar Marburg was third in a family of four sons and two daughters, all of whom—including both parents—have survived him.

In early life, Mr. Marburg showed those characteristics of independence of thought and action that marked his later life. It is related that at the age of five years, by a series of questions which he asked his mother and by reasoning the matter out entirely by himself, he learned to tell the time.

He attended private schools in New Orleans and later in Shreveport, La., and from the beginning showed much aptitude for study, particularly for mathematics. His teacher at Shreveport, Capt. George E. Thacher, soon recognized his mathematical ability and placed him in a class by himself in that subject, with no lesson assignments. Young Marburg would simply report from time to time how far he had progressed in his study. His family have to-day a book in which he kept, in neat, orderly arrangement, solutions of many problems, from simple ones in arithmetic to those in analytic mechanics and calculus. He made it a rule never to ask or accept help on any problem, and this self-reliant attitude was evident throughout his life in everything he undertook.

At the age of fifteen he gave up school and announced his intention of going to college. It was necessary for him largely to work his way through, and he accordingly worked for the next three years in a large wholesale grocery store in Shreveport. He studied at nights to prepare to enter the sophomore class at college. On the advice of Capt. Thacher, and because of his predilection for mathematics, he decided to study Engineering and finally entered Rensselaer Polytechnic Institute in 1882 as a sophomore, at the age of eighteen. The promise of his early youth was fulfilled in his college life, and he was graduated with the degree of Civil Engineer in 1885.

Of his college days the following letters bear testimony to his activi-

* Memoir prepared by Samuel Tobias Wagner, M. Am. Soc. C. E., and Charles Laurence Warwick, Assoc. M. Am. Soc. C. E.

ties and character at that time. Enrique A. Touceda, M. Am. Soc. C. E., writes as follows:

"While he was always intensely interested in Institute happenings and was always popular with the student body, and particularly so with his classmates, he was modest and more or less retiring, having little to do with Institute politics. Without being what is known as a 'grind,' his first thought was to make good in his studies and then to devote as much time as possible to the enjoyment of general college life. During this period little if any attention was paid by any of the students to athletic sports, first due to lack of time, and second to the fact that the Institute possessed no athletic field.

"Marburg as a youth was appreciated by his associates as one of sterling worth, very interesting and very companionable. He was intellectually vigorous and quick of judgment; always dependable and ready to render a friendly service on the instant."

Palmer C. Ricketts, M. Am. Soc. C. E., President of the Institute, writes:

"Marburg, of course, was a very excellent student while here. He was graduated with the degree of Civil Engineer in the Class of 1885. He was a member of the Rensselaer Society of Engineers when a student and was the first editor of the *Institute Quarterly*, a paper which had a brief existence. * * * We all liked him when he was a student here and after he was graduated. He was a very able man, a man of fine integrity and of fine disposition."

He had definitely decided at this time to specialize in structural engineering and aimed to obtain as varied an experience as possible. Consequently, he adopted the plan of going from one bridge or structural company to another, studying at each one until he had the experience he desired. These changes were at times made at a loss of salary, but this feature never swerved him from his determination. In his later associations with young graduates, Mr. Marburg always endeavored to impress on them the importance of gaining experience in their profession and cautioned them against making increase in salary their sole consideration in changing positions.

In 1885 he entered the employ of the Keystone Bridge Company at Pittsburgh, Pa., and remained there until about 1887. The late Edwin Thacher, M. Am. Soc. C. E., who was then Chief Engineer, wrote as follows regarding this engagement:

"I was Chief Engineer of the Keystone Bridge Co. in Pittsburgh in 1885 when Mr. Marburg graduated from the Rensselaer Polytechnic Institute at Troy. He had applied for a position a month or more previously and I told him to come on if he was willing to start in at a salary of $50 per month, which was paid to all beginners. He came at the appointed time, and I set him at work on the design of plate girder bridges, at which he soon became expert. He was kept on this line of work for nearly a year, much longer than he should have been for his own good, for I believe it is the practice in all large bridge

drafting rooms to keep a man at the class of work that he is most expert in; but finally Mr. Marburg before he left did get considerable experience in the design of pin connected bridges and buildings.

"I was very favorably impressed with Mr. Marburg from the first day I saw him. His appearance and bearing were such as to satisfy me that he was a young man of superior ability, and of all the men in the drafting room he was my favorite. He had no bad habits and was a conscientious worker."

From 1887 to about 1889 he was in the service of the Phœnix Bridge Company at Phœnixville, Pa., employed almost entirely in the Computing Department of that company. From 1889 to 1900 he was with the Edgemoor Iron Company, at Wilmington, Del., in the Designing and Estimating Department under C. W. Bryan, M. Am. Soc. C. E., now Chief Engineer of the American Bridge Company.

In 1891 he returned to his home in Shreveport and it was at that time that he met and became engaged to his future wife, Miss Fanny Dulany Moncure. In 1891 and 1892 he was in the employ of the Carnegie Steel Company (Keystone Bridge Company) at Chicago, Ill., under Charles L. Strobel, M. Am. Soc. C. E., who writes as follows as to his work at that time:

"Dr. Marburg was employed by the Keystone Bridge Co. (one of the Carnegie firms), Chicago office, in 1891-92, which office was in my charge. I was at the time Consulting Engineer of the Keystone Bridge Company and of Carnegie, Phipps and Co., Limited. Dr. Marburg was engaged upon the work of preparing shop and mill plans for the structural iron work required for the large fireproof Auditorium Building, Chicago. This was unusually complicated work, as the building was designed to serve several purposes—part to be a concert, opera and convention hall, another part to be an office building and a third part to be a hotel. It was necessary to do the work under great pressure as to time, in order to permit the use of the large hall for the meeting of the National Republican Convention in the fall of 1892. Dr. Marburg was earlier employed in connection with a revised edition of the Carnegie, Phipps and Co. Handbook, then in preparation.

"Dr. Marburg's splendid qualities of conscientiousness, thoroughness, faithfulness and competency were in evidence, as they have so conspicuously been in his later life."

In 1892 a number of engineers recommended him to Dr. Charles Custis Harrison, then Provost of the University of Pennsylvania, for the position of Professor of Civil Engineering at that institution. He was elected Acting Professor of Civil Engineering and a year later became full Professor, a position which he held until his death.

Possibly no one is better qualified to speak of his qualities as a teacher than William Easby, Jr., M. Am. Soc. C. E., who was associated with him for many years at the University of Pennsylvania.

"Few have left a stronger impress of their personality on a life's work than has Edgar Marburg, for twenty-six years Professor-in-charge of the Department of Civil Engineering. Throughout this long period, covering years of devoted service, he gave unstintingly of the best that was in him toward the upbuilding of the department so wisely entrusted to his charge. That he was successful is well known and strikingly apparent to any who can compare the curriculum and facilities for instruction of a quarter of a century ago with those of the present.

"Dr. Marburg's influence on engineering was far-reaching and salutary. It was strongly felt in his work as a teacher, writer and executive throughout a wide and important circle of professional friends and acquaintances. It may truly be said that he supplied in himself, through his broad contact with Engineering, a bond between his students and the professional life which they sought to enter. Though not himself a practicing engineer, he showed in his instruction the fullest appreciation of the value of practical illustration and acquaintance with business procedure. Through patient and sound advice he endeavored at all times to instil into the minds and hearts of his students a high regard for the dignity and worth of their chosen profession. The high standard of professional ethics which he set for them was but the expression of principles which he consistently observed throughout his professional life.

"Dr. Marburg fully recognized and accepted the opportunities and obligations of his position as a teacher in a great institution of learning. With him teaching was not principally an occupation with a livelihood as its object, but a profession of the highest order which he knew to be far-reaching in its influence on the economic and social fabric of the country, worthy of that patient and devoted effort which he always bestowed upon it. His success as a teacher is unquestioned, and followed naturally from the possession of those qualities which secure the respect, confidence and interest of students. He had to an unusual extent the faculty of divesting a subject of difficulties by exposing, through familiar and often homely illustrations, the fundamental thought involved and by arranging related matters with due regard to their importance. Recitations never lagged, were never tedious or unprofitable under the vitalizing influence of his strong nature. He felt the intellectual pulse of his classes and was quick to detect lack of comprehension. It was never too much trouble to extend instruction beyond the class-room to the earnest student in need of assistance.

"A strict disciplinarian in all things essential, Professor Marburg still had the good will if not the affection of his students, based on his absolute fairness and justice and on the evident desire to apply the penalty for misdoing more as a corrective and deterrent than as a punishment. Since his death not a few letters have been received from his former students attributing much of their success to his instruction and influence as a man and expressing heartfelt regret that they had postponed the acknowledgment of their debt to him in person until too late."

The following is a copy of the resolutions of the Faculty of the Towne Scientific School of the University of Pennsylvania upon his death:

"Edgar Marburg, Sc. D., LL. D.,
"Professor of Civil Engineering, 1892-1918.
"Deceased June 27th, 1918.

"Moved by a profound sense of loss, this Faculty deems it a privilege to here record its estimate of the character and attainments of a colleague, who, although removed by death, has left an abiding influence for good through the vitality and idealism of his life work.

"The career of Edgar Marburg, shortened by its very excess of self-giving, nevertheless had long yielded him a distinguished position among engineers and educators. This was due to exceptional force of character and intellectual endowment which, both in preparatory training and professional and teaching activities, placed everything he did on a high plane. His work was marked by clarity and readiness of insight, grasp of essentials and thoroughness of method, and was inspired by high principles and a rugged honesty of view and purpose.

"Efficiency of so high an order, addressed to essentials, through more than thirty years of incessant activity, has made the life work of Edgar Marburg a contribution of positive constructive value to the welfare of the community. In two channels particularly has this been demonstrated; the more widely known that of the American Society for Testing Materials, whose great service to the Engineering and industrial worlds is due, more than to any other single influence, to his labors. The other channel, invisible but potent, is that of the careers of many students who have passed under his influence, carrying away as fundamental to their action his ideals of professional conduct.

"It has been well said of him that he had a constant desire to know the truth and be guided by it; possibly this best explains his life and the reason why we, his colleagues, are moved to add to this summary of his achievements an expression of profound respect for his character and of affectionate regard for the underlying warmth and kindliness of his nature.

"To the family of Professor Marburg we offer our sincere sympathy in their bereavement, and while knowing that no mere words can lessen its magnitude, we yet venture to hope that consolation may be found in the knowledge that his life, although prematurely ended, has been rich in a usefulness not to be measured by length of days.

"*Resolved:* That the foregoing be adopted as the sense of this Faculty, and spread upon the minutes, and that copies be addressed to the family of Professor Marburg and the professional organizations of which he was a member."

In 1906 a new building at the University of Pennsylvania was dedicated for the use of the Departments of Civil, Mechanical and Electrical Engineering, embodying at that time all of the latest approved facilities and apparatus required by technical schools of the highest grade. The planning and detail designing of this building was entrusted to Dr. Henry W. Spangler, at that time Professor of Mechanical and Electrical Engineering, and Dr. Marburg, who were given *carte blanche* by the Trustees. The product of their joint work will always be a monument to each. At the time of the dedication of the building

Honorary Degrees of Doctor of Science were conferred upon Professors Marburg and Spangler.

Almost immediately after settling in Philadelphia, Dr. Marburg became connected with the Engineers' Club of Philadelphia and was its President in 1900. His work in this connection is well set forth by George S. Webster, M. Am. Soc. C. E., as follows:

"In the death of Edgar Marburg, the Engineers' Club of Philadelphia lost one of its most valued members; one who always took an active interest in its welfare.

"He was elected a member of the Club October 21st, 1893, and was chosen one of the Directors in 1895 and 1896; and ably served as President during the year 1900. He was Chairman of the Committee on Public Relations in 1912, and from March 21st, 1908 until the time of his death was one of the Trustees of the Bond Redemption Fund.

"Dr. Marburg was deeply interested in the several movements of the Club to enlarge its sphere of usefulness, to increase its membership and to bring it into closer relations with other technical societies; and he contributed in no small measure to the accomplishment of the splendid results which followed these efforts.

"Earnestness of purpose and high ideals were among his characteristics, impelling him to urge the most exalted standards of ethics for the Engineering Profession, and to recommend as a duty a more active participation by Engineers in public affairs, in order that they might render greater service to the community.

"He was a man of strong convictions, and while energetically presenting his views, was very considerate of the views of others, and heartily united with them in doing that which seemed to be best for the interest of all.

"He took a leading part in making constructive suggestions at the hearings held in October, 1914, before the Engineers' Commission, appointed by the Governor of the State of Pennsylvania to investigate and report upon the advisability of licensing engineers. The clear and logical way in which arguments were presented by him at these hearings exerted a large influence upon the Commission in reaching its decision not to recommend the enactment of a License Law at that time.

"Dr. Marburg was a man of scholarly attainments, and made numerous valuable and interesting contributions to technical literature. He possessed characteristics which made him distinctive among his associates and marked him as a forceful man with original and striking ideas and quick to put them into effect. In fact, it was his unbounded energy coupled with the deep knowledge of the subjects he took hold of—and his resourcefulness in solving the problems which arose therewith—which made of him a distinguished and representative member in his profession.

"His sterling qualities and unquestioned honesty of purpose gained for him the respect and admiration of all his associates, and his death is mourned as a great personal and professional loss not only by the members of the Engineers' Club of Philadelphia but by all who had the privilege of his acquaintance."

At a meeting of the Board of Directors of the Engineers' Club held July 8th, 1918, the following resolution was adopted:

"The Engineers' Club of Philadelphia learns with profound sorrow of the death, on June 27th, 1918, of its beloved fellow-member and Past-President, Edgar Marburg. The Engineers' Club has, in the death of Edgar Marburg, lost one of its most valued members, whose loyal service has left a deep impression on the history and development of the Club, whose scholarship has earned for him an international reputation and whose personal influence has helped the members in their work and professional advancement.

"The Engineers' Club extends to his family its warmest sympathy in their bereavement and sorrow; and as an acknowledgment of his loyalty, brilliant leadership, kindly sympathy, ready helpfulness and professional ability, it is directed that this resolution of condolence and regret be published in the Journal of the Engineers' Club, together with a sketch of his life."

Possibly the greatest work in which he was ever engaged was his connection with the American Society for Testing Materials. In 1898 he joined the American Section of the International Association for Testing Materials and helped to organize the American Section of that Association. In 1902, when it became evident that a real leader would have to be found to vitalize the Society, the American Section turned to him. A concise statement of this part of his work is given in the following minute on his death which was passed at the October meeting of the Executive Committee of the Society, in 1918:

"In the death of Edgar Marburg, which occurred on June 27th, 1918, the American Society for Testing Materials has suffered the irreparable loss of one of its most eminent and distinguished members; its executive officer since its incorporation; a leader in engineering thought and education and in the field of testing materials.

"He was one of a small group which, in 1898, organized the American Section of the International Association for Testing Materials. In 1902 he was appointed acting secretary of the Section. He at once prepared for the Executive Committee a clear, comprehensive statement of the purposes of a testing society, as he conceived them, which led to the incorporation of the Society as an independent body. He was elected Secretary and Treasurer of the new society, and to him was entrusted the executive direction of its affairs. From then till his death, a period of sixteen years, he worked assiduously for the success and advancement of the Society. He devoted much time and thought to the work of its technical committees and the standardization of specifications and methods of tests of the materials of engineering, and he shaped the policy under which these essential activities of the Society are conducted. The marked success which has attended the work and growth of the Society, and the interest and pleasures of its annual meetings, have been due in a large part to his tireless and painstaking efforts. To his labors he brought great executive and

organizing ability, excellent judgment, a clear grasp of the many and varied problems which confronted him, a sympathetic understanding of humanity, tremendous enthusiasm and zeal, and thoroughness in everything he undertook. His high ideals were reflected in every word and act. He was ever zealous of the good name of the Society and sought throughout his leadership to extend its usefulness in every proper field.

"To the officers and members of the Executive Committee, who were privileged to work intimately with him, his death brings a deep sense of personal loss and grief. His high integrity, his rugged honesty, his open-mindedness, and his evident sincerity of purpose, endeared him to his associates, and commanded their admiration, respect and devotion.

"The character and personality of Edgar Marburg have indelibly impressed themselves upon the American Society for Testing Materials —an inspiration to the Society for all time."

Past-President A. A. Stevenson of the American Society for Testing Materials has kindly prepared the following statement which shows many of the characteristics of the man:

"No doubt the members at large of the American Society for Testing Materials appreciate, in a certain measure, how much Dr. Marburg's connection with the Society had to do with its success. Those of us who as officers of the Society and members of the Executive Committee during Dr. Marburg's tenure of office, had the privilege of working with him, can alone have a full realization of the extent to which his character as a man, with his ideas and ideals, independent of his work, had to do with the growth of the Society in influence and in numbers.

"Any one working with the Doctor could not but have the greatest admiration for him. While at times one might have differed with him, there was never a question as to his motives, or a thought that he was prompted by any other motive than what, in his mind, was best for the Society. Dr. Marburg had at all times the interest of the Society at heart, and no one could have been more jealous of its good name. He would make any sacrifice and go to any length in working to advance its interest. His very enthusiasm and devotion led to overwork, and I have no doubt that this, in connection with his work at the University, was largely one of the causes that led to his untimely death in the prime of life. With it all, one could not but be impressed by the great modesty of the man. He was most bitterly opposed to anything that might seem like self aggrandizement. It was the Society he thought of, and not himself. His desire was to see results; his greatest satisfaction in work accomplished.

"As an officer and member of the Executive Committee I worked with Dr. Marburg seven years; saw a great deal of him, and had many conferences concerning the work and the future of the Society. The official relationship developed into a warm personal friendship that is, and always will be, one of my most cherished memories. His 'Old man, how are you?' over the telephone or when met, is sadly missed.

"To me the most outstanding trait in Dr. Marburg's character was his sterling honesty. He was honest in his ideals, in his work, and in his life. What more fitting tribute could be paid to a man?"

At the Annual Meeting of the American Society for Testing Materials, held in Atlantic City, N. J., in June, 1919, a special Session was held in his honor and addresses were made as follows:

Introductory Remarks.....President G. H. Clamer
Contributions to Engineering.....Mr. John M. Goodell
His Early Activities in the Society.....Mr. Robert W. Lesley
His More Recent Service in the Society....Prof. A. N. Talbot
A Personal Tribute.....Captain Robert W. Hunt

The addresses in full can be found in the *Proceedings* of the American Society for Testing Materials for 1919.

In 1919 the American Society for Testing Materials established an Edgar Marburg Memorial Scholarship in the University of Pennsylvania, to be awarded by the University to a student in the Department of Civil Engineering. The fund for this scholarship ($5 000) was raised by subscription from the members of the Society.

With reference to the above *Engineering News-Record* commented editorially as follows:

"No more appropriate form of memorial to the memory of Edgar Marburg than the proposed permanent scholarship in his own institution, the University of Pennsylvania, to which he gave practically the whole of his working life, could be conceived. The plan by which the American Society for Testing Materials is collecting the fund for a memorial scholarship, is worthy of the highest commendation. This form of recognition would appeal to Dr. Marburg himself, because his whole heart went out in the true teaching instinct to develop young engineers into men worthy of the great profession to which he gave his life. It is to be hoped that the future students who will benefit by this scholarship may fully appreciate the honor and repay it a hundred fold by their later contributions to engineering progress. It is unfortunate that only members of the A. S. T. M. have been asked to subscribe. There is no doubt whatever that thousands of other engineers the country over would be glad to add their tribute to a man who had been their inspiration and guide either in the days of their early training or in their engineering experience."

He acted as Consulting Engineer at various times, notably in the preparation of the design for the Rock Creek Bridge on the line of Massachusetts Avenue, Washington, D. C., under commission for the Corps of Engineers, U. S. A.

In 1904 he was elected Lecturer on Structural Engineering at the Franklin Institute, and he was Past-Chairman of its Committee on Science and the Arts. He was also Secretary of the American Society

for the Promotion of Engineering Education, and Past-President of the honorary society, Sigma Xi.

In addition to a number of papers for the American Society for Testing Materials, his best known contribution to engineering literature is his work on "Framed Structures and Girders—Theory and Practice. Volume 1". This was the first of an unfinished series of three volumes covering the field of structural steel design and erection.

In 1896 he presented a paper before the Engineers' Club of Philadelphia, on "Cantilever Bridges". This notable paper was used in his classes from the time of its presentation.

He also contributed papers to the American Society for the Promotion of Engineering Education. In 1914 he was honored by Franklin and Marshall College with the degree of Doctor of Laws and Letters.

He was married to Miss Fannie Dulany Moncure of Shreveport, La., at Shreveport, on August 14th, 1893. Mrs. Marburg and their four children, Edgar, Mrs. Samuel T. Ziegler, Clara, and Anita survive him.

He was elected a Member of the American Society of Civil Engineers on October 6th, 1897.

WILLIAM AUGUSTUS MONCURE, M. Am. Soc. C. E.*

DIED OCTOBER 3D, 1918.

William Augustus Moncure, son of the late Thomas Jefferson Moncure and Margaret Elizabeth Moncure, was born near Falmouth, Va., on February 26th, 1870.

After completing his studies in the Suffolk Military Academy, he entered the Virginia Military Institute, from which he was graduated in 1889. After his graduation, he immediately began to engage actively in his chosen profession as a Civil Engineer.

During the summers of 1886 to 1889, he was employed as Rodman and Instrumentman by the Seaboard Air Line Railway. From October, 1889, to September, 1891, he was with the Richmond and Danville Railroad, as follows: From October to December, 1889, as Draftsman in the Chief Engineer's office; from December, 1889, to May, 1890, as Leveler and Transitman; and from May, 1890, to September, 1891, as Resident Engineer in charge of construction.

From February to October, 1892, Mr. Moncure was Assistant to the Engineer in charge of construction on the Ohio Extension of the Norfolk and Western Railroad; and from November, 1892, to September, 1893, he was Levelman and Transitman in charge of change in line and construction work on the Philadelphia Division of the Pennsylvania Railroad.

From December, 1893, to March, 1894, he was engaged in the Department of Sewers, Washington, D. C., and, during the latter month, with the Pennsylvania Railroad. Then, until April, 1895, he was engaged on resurveys of portions of the Monongahela, Maryland, and New York Divisions, and from April, 1895, to April, 1897, he was Principal Assistant to the Engineer in charge of important changes on the Philadelphia and New York Divisions.

In May, 1897, Mr. Moncure was appointed Assistant Engineer in the Engineering Department of the Pennsylvania Railroad, and in January, 1900, was transferred to the Real Estate Department, where, until January, 1902, he was in charge of the engineering and drafting force. Then, until June, 1906, he was Principal Right of Way Agent, and in January, 1907, was appointed Engineer of Right of Way. In January, 1913, he was made Assistant Real Estate Agent, and in July, 1918, Real Estate Agent.

In getting off a train at Fredericksburg, Va., Mr. Moncure was injured seriously, and died in the Mary Washington Hospital at Fredericksburg, on October 3d, 1918.

* Memoir prepared by H. E. Tripler, Esq.

On November 6th, 1895, Mr. Moncure was married to Miss Caroline Ashe Pemberton, of Albemarle, N. C., who, with three daughters and four sons, survives him.

Mr. Moncure was elected a Junior of the American Society of Civil Engineers on June 21st, 1894, an Associate Member on October 7th, 1896, and a Member on June 5th, 1906.

AUGUSTUS MORDECAI, M. Am. Soc. C. E.*

DIED JULY 28TH, 1918.

Augustus Mordecai, son of the late Maj. Alfred Mordecai and Sara Hays Mordecai, was born in Philadelphia, Pa., on September 8th, 1847.

He attended the Polytechnic College of Pennsylvania, from which he was graduated in June, 1866, and then, for about a year, served as a Rodman on the construction of the Dutchess and Columbia Railroad, now a part of the Central New England System.

During the next four years Mr. Mordecai was in charge of construction divisions, successively, on the Connecticut Western Railroad (now also a part of the Central New England System), on the St. Louis, Council Bluffs, and Omaha Railroad (now a part of the Missouri Pacific System), and on the Pittsburgh, Virginia, and Charleston Railroad (now a part of the Pennsylvania System), remaining on each work until its completion.

In 1872, Mr. Mordecai went to Cleveland, Ohio, as Division Engineer in charge of maintenance on the Atlantic and Great Western Railroad, which was afterward merged into the present Erie Railroad System. For twenty years he remained in Cleveland, associated with the Erie road and its successors, filling the positions, successively, of Division Engineer, Resident Engineer, and General Roadmaster. In 1892, he was appointed Chief Engineer of the Erie System, with headquarters in New York City, and served in that capacity until 1896. For ten years thereafter he resided in Cleveland as Assistant Chief Engineer in charge of construction and maintenance on all the lines of the Erie System from Salamanca to Chicago, and was then appointed Chief Engineer of the Lorain and Ashland Railroad, in charge of the location and construction of that line. He was afterward associated with J. F. Stevens, M. Am. Soc. C. E., as Office Engineer in the valuation of the New York, New Haven, and Hartford Railroad, until the completion of that work.

From 1909, Mr. Mordecai was active in Cleveland as a Consulting Engineer, notably in connection with the Belt Line around Cleveland, and with valuations of the properties of the New York Central, the New York, New Haven, and Hartford, the Lehigh Valley, and the Canadian Pacific Railroad Companies, and also the Detroit United Street Railway Company. Up to the time of his fatal illness, he was engaged in similar work for the Erie Railroad Company. In 1912, he was appointed, by the Mayor of Cleveland, a member of that city's River and Harbor Commission.

In Cleveland, on October 14th, 1897, Mr. Mordecai was married to Miss Margaret Bowman, who survives him.

* Memoir prepared by John C. Trautwine, Jr., Assoc. Am. Soc. C. E.

Mr. Mordecai's kindly and unassuming nature, his sterling integrity, and his fine sense of humor endeared him to all who had the good fortune to know him.

Mr. Mordecai became a Member of the American Railway Engineering Association in 1909, and was one of the original members of the Cleveland Engineering Society. He was elected a Member of the American Society of Civil Engineers on February 1st, 1893, and served as a Director for the period 1895-97.

CHARLES JOHN AUGUSTUS MORRIS, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1918.

Charles John Augustus Morris, the son of Charles A. F. and Mary Ellen Morris (*nee* Lane), was born in Brooklyn, N. Y., on July 26th, 1850.

His early education was obtained in public and private schools, his professional education and training being acquired later under the tutorage of his father, a graduate of Dublin University and a prominent and experienced civil engineer and contractor, who, for many years, was one of the leading pioneer civil engineers in railway construction work in the earlier days of the Northwestern States.

So far as can be learned, the first important engineering work performed by Charles John Augustus Morris consisted of the survey and location, in 1871, of an extension of the St. Paul and Pacific Railroad from a point now occupied by Barnesville, Minn., to St. Vincent, on the northern boundary of the State. In 1875 and 1876, Mr. Morris was similarly engaged on surveys for the St. Paul and Pacific Railroad in the western part of Minnesota.

In 1878, he was engaged on Government work at Rock Island, Ill., and, during 1879, was employed in the United States Government Engineer's office in St. Paul, Minn., under Maj. Allen, of the U. S. Corps of Engineers. In 1880, Mr. Morris had charge of the construction of a water-power canal for the Pillsbury "A" Mill, at the Falls of St. Anthony, in Minneapolis, Minn.

In 1881, he was employed by the St. Paul, Minneapolis and Manitoba Railway Company on bridge, trackage, and other construction work, and in 1882, 1883, and part of 1884, he made the surveys and located the present Stone Arch Bridge across the Mississippi River, for the Minneapolis Union Railway Company. During the construction of this noteworthy bridge, he was the Supervising Engineer in charge, the Chief Engineer being the late Col. C. C. Smith, M. Am. Soc. C. E., of the Great Northern Railway Company. During this period Mr. Morris was also in charge of the re-arrangement of trackage for the new Union Station in Minneapolis and the construction of the buildings forming that station; he remained in the employ of the Great Northern Railway Company until 1887.

From 1888 to 1890, Mr. Morris was a member of the contracting firm of McMullen and Morris, engaged in heavy masonry work, particularly in constructing large retaining walls in St. Paul and Minneapolis in connection with the lowering of track grades. During this time, he also had charge of the work of erecting the foundation piers for the High Bridge in St. Paul.

* Memoir prepared by William de la Barre, M. Am. Soc. C. E.

On March 22d, 1888, he was appointed Chief Engineer of the St. Paul and Duluth Railway Company, and continued in that position until June 15th, 1890, at which time the property was taken over by the Northern Pacific Railway Company. Mr. Morris then entered the construction field as a member of the firm of Frankman Brothers and Morris.

During 1891 and 1892, he was Chief Engineer of the Minneapolis Western Railway Company, having charge, among other matters, of the design, construction, and erection of the steel railway bridge over the Mississippi River immediately below the Falls of St. Anthony. During this period he also acted as Consulting Engineer for the Northwestern Fuel Company in connection with the design and construction of its coal dock substructure at Superior, Wis. The firm of Frankman Brothers and Morris also constructed the timber cribbing and timber coal bins of Section No. 1 of the Northwestern Fuel Company's Superior Dock No. 1 on Connor's Point, as well as the timber cribbing for the Company's Coal Dock No. 2 on St. Louis Bay, near Duluth.

From September, 1895, to January, 1898, Mr. Morris was engaged in superintending the construction of the St. Anthony Falls Water-power Company's lower dam and power house, including the hydro-electric development of 10 000 h. p., below the Falls of St. Anthony.

From 1900 to 1904, he was actively employed in various engineering and construction works at the head of Lake Superior, particularly at Duluth, Two Harbors, and Superior, as Chief Engineer of companies handling coal and iron ore, continuing in that capacity until the Northwestern Fuel Company's property was acquired by the Pittsburgh Coal Company. It was under his direction and superintendence, that the first hydraulic dredge on the Great Lakes was constructed and put in operation. During this period he designed and supervised the construction of Dock No. 5, at Duluth, for the Pittsburgh Coal Company, with a storage space of 400 by 1 000 ft. with forty stationary hoists. Later, this storage space was lengthened to 1 360 ft., and four movable hoists were added to the equipment. In 1906, the firm of Frankman Brothers and Morris constructed the timber cribbing and anthracite storage building, of 2 000 000 tons storage capacity, on Section No. 2 of the Northwestern Fuel Company's Dock No. 1 at Superior, Wis.

From July, 1907, to June, 1908, Frankman Brothers and Morris was engaged in the erection, for the Northern Pacific Railway Company, of its Valley City, N. Dak., viaduct, which is one of the largest and heaviest viaducts in the Northwest, being about 3 000 ft. long and 150 ft. high, with double-track towers and substructures, the erection of which covered the placing of more than 7 000 tons of steel.

In 1907, Mr. Morris was also employed as Consulting Engineer and had charge of the construction of Dock No. 1 of the Berwind Fuel

Company at Superior, Wis., and, at about this period, he was also engaged in the construction of additional dock facilities at both Superior and Duluth.

In 1909 he became the senior partner of the contracting firm of Morris and Shepard (changed in 1912 to Morris, Shepard and Dougherty), which firm was subsequently engaged in various railway construction work for the Chicago, Milwaukee and St. Paul Railway Company, the Chicago, Burlington and Quincy Railway Company, and the Great Northern and Northern Pacific Railway Companies and, in addition, in dock construction work at the head of the Lakes.

At the time of his death, the firm of Morris and Dougherty (Mr. Shepard having retired) and the George J. Grant Construction Company had the contract with the Union Depot Company of St. Paul, Minn., for the design and construction of the new Union Station in that city and the building of the various adjacent yard systems connected therewith.

Mr. Morris was married, on December 31st, 1878, at Cleveland, Ohio, to Sarah R. Saunders, who survives him; other surviving relatives are two brothers and two sisters.

He was a member of the Civil Engineers Club of St. Paul, Minn., Minnesota Club, St. Paul Athletic Club, University Club, and Town and Country Club, all of the same city.

Mr. Morris was a gentleman with whom it was a genuine pleasure to be acquainted and associated. His ideals were always of the highest character, and, equally as a friend or business associate, he was thoroughly faithful, conscientious, and reliable, and was held in high esteem by employer and employee alike. He was energetic and resourceful to a high degree, and his ability as an engineer, together with his pleasing and tactful personality, contributed largely in insuring the success of his business life.

Mr. Morris was elected a Member of the American Society of Civil Engineers on October 3d, 1883.

HENRY PRENTICE MORRISON, M. Am. Soc. C. E.*

DIED DECEMBER 17TH, 1918.

Henry Prentice Morrison was born in Troy, N. Y., on January 14th, 1857. At an early age he came to New York City with his parents and settled on the lower west side, where he lived for many years. He was educated in the public schools, and was graduated from New York University in 1880, with the degrees of B. Sc. and C. E. For a short time after his graduation he served as Assistant to Mr. George N. Harris, on the Baltimore and Ohio Railroad.

In March, 1881, Mr. Morrison entered the Department of Public Works of the City of New York, where he remained until 1893. During these 12 years he served as Assistant to the late Stevenson Towle, M. Am. Soc. C. E., in the Bureau of Sewers; in the Bureau of the Water Purveyor; and with Horace Loomis, M. Am. Soc. C. E., as Principal Assistant in charge of establishing lines and grades, and the construction of pavements.

In 1889, under a special repaving law, he had charge of all repaving work in New York City below 14th Street, including stone block, asphalt, brick, Telford, and wooden pavements.

In the early part of 1893 Mr. Morrison was appointed by the Supervisors as Engineer of County Roads for the County of Richmond, and, as such, under what was known as the County Road Law, had charge of the preparation of surveys, plans, and specifications for the construction of about 100 miles of modern macadam and block pavements. Of these, 55 miles were completed under his supervision, and, in addition, he designed and superintended the construction of five masonry, brick, and steel highway bridges, two steel swing-bridges, and a wooden trestle roadway, 1 800 ft. long.

He also had charge of the building of 30 miles of trolley railroads, heavy retaining walls, sewers, dock improvements, and a water supply system for the county buildings. In addition, as County Engineer, he had charge of the maintenance of all county roads.

Mr. Morrison served in this position with distinction until consolidation with the City of New York, on January 1st, 1898, when he automatically became connected with the new City Government. He was appointed, by Mayor Van Wyck, Deputy Commissioner of the Departments of Highways, Sewers, and Water Supply, and Chief Engineer of the two first-named Departments. This combining of five positions under one official was done for reasons of economy, as it was deemed extravagant at that time to have three fully developed departments in the Borough of Richmond. For four years, or until the begin-

* Memoir prepared by Theodor S. Oxholm, M. Am. Soc. C. E.

ning of 1902, he had charge of all improvements and maintenance work in the Borough of Richmond. On January 1st, 1902, the revised Charter of the City went into effect, giving each Borough autonomy under its Borough President, and abolishing the former departments. After reorganizing the old departments into bureaus, and placing the construction work, maintenance, and Bureau of Street Cleaning on a good working basis, which he did at the request of Borough President George Cromwell, he separated from the City service and entered into private practice, with offices in the Park Row Building, New York City.

For the next 13 years Mr. Morrison prosecuted a general engineering practice, and in 1913 and 1914 he was specially engaged by the Governor of New York to investigate the frauds in the building of State roads. He was also appointed City School Commissioner in 1912, and served for two years. In September, 1915, he was appointed Commissioner of Public Works for the Borough of Richmond by President Calvin D. Van Name, in which position he served until his death.

Mr. Morrison's skill and experience as a road engineer will be greatly missed in New York City. His ability as a public speaker and his great interest in all educational matters made it impossible to have a successful meeting, public dinner, or entertainment, in the Borough of Richmond, without his active presence.

His friends remember him as a man of most pleasing personality, a ready wit, most entertaining on all occasions, an astute politician, and one whose advice was much sought. His associates in the Department of Public Works miss his sound and practical judgment and advice on important matters.

On April 30th, 1890, he was married to Miss Belle A. Von Eiff, of Brooklyn, N. Y., who survives him.

Mr. Morrison was elected a member of the American Society of Civil Engineers on April 6th, 1898.

CLARENCE RUFUS NEHER, M. Am. Soc. C. E.*

DIED MAY 13TH, 1918.

Clarence Rufus Neher was born in Ashokan, Ulster County, N. Y., on February 27th, 1860. He was educated in the public schools at Ashokan, and at the Academy at Nassau, N. Y.

About 1881, Mr. Neher joined the Engineering Corps of the Genesee Valley Canal Railroad, at Nunda, N. Y., now the Rochester Division of the Pennsylvania Railroad. In 1884 he accepted the position of Assistant Engineer on the Washington and Ohio Railroad, in Virginia, under Robert Bell, Chief Engineer. Between 1887 and 1889 he was employed as Engineer in the Maintenance of Way Department of the Richmond and Danville Railroad, with headquarters at Greensboro, N. C. In 1889 he became Engineer in the Maintenance of Way Department of the Western New York and Pennsylvania Railroad, now the Rochester Division of the Pennsylvania Railroad, with headquarters at Rochester, N. Y. He held this position about nine years, or until 1898, and then resigned to become Resident Engineer in the New York State Engineer's Department on the Western Division, with headquarters at Rochester, N. Y.

About 1900 Mr. Neher resigned this position and was appointed Engineer for the Donnelly Construction Company, of Buffalo, N. Y., remaining with the Company while the foundations for the Great Eastern and Dakota grain elevators, and other important work near Buffalo, were built. In 1901 he became Engineer and General Manager of the Continental Engineering and Contracting Company, of Buffalo, N. Y., and, in this capacity had charge of building the concrete foundations and floors of the grain elevator of the Board of Harbor Commissioners, at Montreal, Que., Canada, and also a large grain elevator at Fort William, Ont., Canada. He also had charge of building all the concrete structures on the Buffalo and Susquehanna Railroad, and the concrete dam across the Cumberland River at Burnside, Ky.

About 1909 Mr. Neher was made Engineer for the Atlantic, Gulf and Pacific Company and had charge of the engineering work on 12 miles of Barge Canal contracts between Smith's Basin and Whitehall, N. Y.

About 1912 Mr. Neher entered the service of the Buffalo Dredging Company, of Buffalo, N. Y., as Engineer and General Manager. While with this company he had charge of completing the large Barge Canal Dam at Hinckley, N. Y., and constructing the new double-track line of the International Railway Company, between Buffalo and

* Memoir prepared by Henry A. Van Alstyne, M. Am. Soc. C. E.

Niagara Falls, N. Y. He was in the employ of this company at the time of his death.

Mr. Neher was a very capable and energetic man, of high character and pleasing personality. He was married in 1882 to Nellie Louise Dodge, of Mount Morris, N. Y., who died in 1913, leaving two sons, Carl Clarence and Roy Herbert Neher. In February, 1917, he was married to Miss Janet Alison Carmichael, of Albany, N. Y., who survives him.

Mr. Neher was elected a Member of the American Society of Civil Engineers on June 4th, 1902.

WILLIAM NEWBROUGH, M. Am. Soc. C. E.*

DIED NOVEMBER 2D, 1919.

William Newbrough was born at Philadelphia, Pa., on December 15th, 1860. After receiving his preparatory training at Dr. Sach's School, he entered Columbia University, New York City, in 1876, and was graduated from the School of Mines in 1884 with the degree of E. M. He received his A. M. degree in 1885.

From 1884 to 1886, he was Instructor in Civil Engineering at Lafayette College, and from 1886 to 1888 acted as Instrumentman and Bridge Engineer with the Atchison, Topeka and Santa Fé Railroad.

During 1888-89, Mr. Newbrough served as Professor of Engineering in the State College of Kentucky, and for the two years following he was in private practice and also held a position as Computer in the United States Surveyor-General's Office, at Salt Lake City, Utah. He continued private practice in the latter city until 1900, including among his more important work, Contract 188 for the survey of the Public Lands of the United States in Utah, the duties of Chief Engineer of the Lucerne Land and Water Company, and the design, location, and construction of 14 miles of a canal crossing three divides and costing $23 000.

In 1892 Mr. Newbrough designed and located the Black's Fork and Twin Butte Canals at Fort Bridges, Wyo., which were enlarged, later, partly by himself. From 1894 to 1896 he was engaged on surveys, statistics, tracing coal veins and outcrops, general exploratory work, assessments, etc., for Mr. P. J. Quealy of Kemmerer, Wyo., in connection with the development of the Kemmerer coal deposits, and from 1896 to 1900, he was employed as Engineer for Quealy and Kemmerer and the Kemmerer Coal Company, having full charge of all engineering work. He also laid out the Town of Kemmerer and designed its waterworks.

In 1900, Mr. Newbrough went to Detroit, Mich., as Acting Assistant Superintendent of the Penberthy Injector Company. In 1902, he returned to Evanston, Wyo., and immediately was engaged as Consulting Engineer by the Kemmerer Coal Company. In 1903 he was appointed Engineer of the Diamond Coal and Coke Company, Diamondville, Wyo., and from 1902 to 1904 acted as Engineer for Beckwith, Quinn, and Company, Evanston, Wyo.

In 1905, Mr. Newbrough engaged in private practice as a Civil and Mining Engineer with an office at Evanston, Wyo., until 1914, when he returned to Detroit, Mich., as General Superintendent of the Penberthy

* Memoir prepared from information on file at the Headquarters of the Society.

Injector Company. He remained with this company until 1916, when he opened an office as a Consulting Engineer at Kemmerer, Wyo., which he continued until his death on November 2d, 1919.

Mr. Newbrough was married to Gertrude Sturtevant, of Brooklyn, N. Y., in January, 1897. She died in 1917. He is survived by a sister, Elizabeth Newbrough.

Mr. Newbrough was elected a Member of the American Society of Civil Engineers on April 6th, 1904.

LEWIS ABEL NICHOLS, M. Am. Soc. C. E.*

DIED MARCH 5TH, 1919.

Lewis Abel Nichols was born in Florence, Italy, on August 26th, 1851, of American and English parents. After his mother's death in 1856, his father returned to the United States with his family, and settled in Danvers, Mass. He died in 1858, and Mr. Nichols was left an orphan at the age of seven.

After attending the public schools and High School at Danvers, he entered the Massachusetts Agricultural College at Amherst, Mass., in 1867, and was graduated in 1871, in the pioneer class of that institution.

Having chosen Civil Engineering as a profession, Mr. Nichols began his life work in the fall of 1871, by laying out town lots in an addition to the City of Fall River, Mass. During the latter part of the same year, he was employed as a Leveler on the location of the Massachusetts Central Railroad. In 1872 he was placed in charge of the party and finished the location and construction of a Division of the railroad in the fall of 1873.

In 1874 and 1875, Mr. Nichols was engaged in private practice in Eastern Massachusetts. He was elected City Engineer of Chelsea, Mass., in 1875 and retained this office until 1877, when he resigned to take charge of the Marblehead Water-Works. In 1878 he resumed his private practice, maintaining an office in Boston for two years.

During 1880 and 1881, Mr. Nichols was a Transitman on reconnaissance, preliminary, and location surveys in New Mexico and Arizona, for the Atchison, Topeka and Santa Fé Railroad. In the latter year, he was appointed Division Engineer on the California Southern Railroad, and was in charge of some particularly heavy and difficult work in the Temecula Canyon, California. During 1883 and 1884, he was employed as Division Engineer on the Tampico Division of the Mexican Central Railroad, in charge of the location and construction of more than 100 miles of heavy mountain road.

Mr. Nichols was Engineer of Buildings on the Chicago, Burlington, and Northern Railroad in 1886, and, in the spring of 1887, became Locating Engineer for that road on projected branches. He was Assistant Chief Engineer on the Illinois Valley and Northern Railroad in 1887 and 1888, and, in the latter year, Locating Engineer on the Oregon Railway and Navigation Company's extension through Idaho. In 1889 he was Office Engineer on the Toledo, St. Louis and Kansas City Railroad, and, in 1890, built some exceptionally heavy and difficult work on the Cumberland Valley Extension. During 1891 he served as Locating Engineer on the Toledo and Chicago Bee Line, but it was not built. Mr. Nichols then became Chief Engineer of the

* Memoir prepared from information on file at the Headquarters of the Society.

La Salle and Bureau County Railroad, a belt line to connect the Illinois Central, Chicago, Burlington, and Quincy, and the Chicago and Northwestern Railroads in the vicinity of La Salle, Ill.

The foregoing gives some indication of the extensive and responsible railroad work done by Mr. Nichols. In 1908, he gave up the active practice of his profession, except as a Consulting Engineer, and devoted most of his time to the interests of the Chicago Steel Tape Company, a corporation which he organized in 1903, and of which he was President at the time of his death.

Mr. Nichols was married in 1879, to Miss Carrie W. Putnam of Danvers, Mass. They had no children.

He was a member of the Woodlawn Park Lodge, A. F. and A. M., of Chicago. He was also a member of the Western Society of Engineers and several other technical societies.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on October 5th, 1892.

ALEXANDER JOSEPH NORRIS, M. Am. Soc. C. E.*

DIED MAY 30TH, 1918.

Alexander Joseph Norris was born on November 4th, 1864, at Vera Cruz, Mexico, where his father, Henry De Butts Norris, one of the early associates of William R. Grace in railway construction in South America, was engaged in building a railroad. The boy studied under private tutors, but was sent to the United States to complete his education, and was graduated from Rensselaer Polytechnic Institute in June, 1886, with the degree of Civil Engineer.

Mr. Norris began his engineering career as Draftsman for the Phœnix Bridge Company, at Phœnixville, Pa., and was engaged in that capacity and also on special bridge inspection for M. S. Carter and Company, of St. Louis, Mo., and for the Missouri Pacific Railroad Company, from November, 1886, to September, 1887.

From September, 1887, to March, 1889, he was employed by the City of New York as Inspector on street paving, and, after April, 1888, was in charge of all paving for the city north of 59th Street and east of Fifth Avenue. He then went to the Connecticut River Granite Company, of Lyme, Conn., as Superintendent, which position he held until June, 1890, when he was appointed Assistant Engineer on his father's staff with the Peruvian Corporation, Limited, of London, and made reconnaissances and preliminary surveys and estimates for railways in Peru and Bolivia, as well as examinations and reports for the reconstruction and equipment of practically all the railroads in Peru.

In 1891, Mr. Norris was appointed first as Engineer on the location of a 50-mile extension of the Southern Railway (Ferrocarril del Sud) and afterward as Chief Engineer of Construction, designing all bridges, culverts, stations, etc.

From July, 1893, to November, 1894, he was employed as Assistant Engineer with the City of New Orleans, on drainage work for that city. While in this position illness in the family compelled Mr. Norris to resign and take charge of his father's business affairs, in which he continued until June, 1896. He then engaged in private practice as a member of the firm of Pratt and Norris, of Essex, Conn., quarry operators and general contractors for the construction of breakwaters, buildings, and all classes of stone work.

In November, 1897, Mr. Norris was appointed Assistant Engineer on the Nicaragua Canal Commission with which he remained until March, 1901, serving for 10 months in Nicaragua as Transitman and Chief of Party, and the remainder of the time in Washington, D. C., on estimates, drafting plans, etc.

* Memoir prepared from information on file at the Headquarters of the Society.

In March, 1901, Mr. Norris again engaged in the private practice of engineering as senior member of the firm of A. J. Norris and Company, Contractors, which firm built part of the electric railway between Amsterdam and Schenectady, N. Y. In November, 1901, he organized the firm of Norris and Penguet, and was engaged on contract work for the Chesapeake and Ohio Railroad on the Big Sandy River in Kentucky, as well as for the Deepwater Railroad in West Virginia.

In March, 1906, Mr. Norris went to Peru as Chief Engineer of the Ferrocarril Central del Peru, and remained in that country until his death on May 30th, 1918. During this time he served as Engineer in charge of a 60-mile extension of the Central Railway of Peru; Engineer in charge of the Chilete Extension of the Pacasmayso and Guadalupe Railway; and as Chief Engineer in charge of all construction and maintenance for the Peruvian Corporation, Limited, of London, which position he held at the time of his death.

The greatest railway construction project with which Mr. Norris was connected, and in which his engineering skill was shown, was the successful construction of the railroads, reaching an altitude of practically 16 000 ft., over the Peruvian Andes, and their subsequent maintenance and improvement, which work brought him international fame.

Mr. Norris was married in November, 1893, to Frederika H., the only child of Capt. Henry C. Henshaw, U. S. A. (*Retired*), who, with two sons and a daughter, survives him.

In recognition of his work in Peru, Mr. Norris was the recipient of many honors, among which were medals awarded by various scientific societies and by the Government of Peru. He was frequently sent as a representative, both by the Peruvian Government and the Peruvian Corporation, Limited, on important missions to Europe and the United States, and was widely known in the Latin-American countries, where the news of his death was received with sincere regret and sorrow by his many friends.

Mr. Norris was elected a Member of the American Society of Civil Engineers on October 3d, 1911.

JOHN PATRICK O'DONNELL, M. Am. Soc. C. E.*

DIED DECEMBER 2D, 1919.

John Patrick O'Donnell was born in Dublin, Ireland, on July 1st, 1859.

He was articled with Messrs. Stevens and Sons, Railway Signaling Engineers, of London and Glasgow, in 1875, and served with that firm until 1882, when he was appointed Assistant Signal Superintendent on the Lancashire and Yorkshire Railway.

From 1885 to 1889 Mr. O'Donnell was in the service of the London and South Western Railway, under Mr. Jacomb, Chief Engineer, and during this period he designed and carried out extensive alterations to the signaling system at Waterloo Station in London, and other works, for that company. Subsequently, he joined Mr. Dutton in establishing the firm of Dutton and Company, Railway Signaling Engineers, of London and Worcester, leaving the firm in 1894 to join the late Mr. A. G. Evans, in founding the firm of Evans, O'Donnell and Company, Railway Signal Works, Chippenham and London, which carried out many extensive signaling works in England and abroad. Evans, O'Donnell and Company subsequently amalgamated with Saxby and Farmer, Limited, and Mr. O'Donnell held the position of Joint Managing Director for a number of years.

In 1901 he formed the British Pneumatic Railway Signal Company, of which he was Chairman and Managing Director until his death. Mr. O'Donnell started the first works for the manufacture of pneumatic tools on a considerable scale in Great Britain, and was one of the pioneers of power signaling there. He installed the first automatic signals on main passenger lines, *viz.*, on the South Western Railway between Grateley and Andover, and subsequently between Woking and Basingstoke. The first power signals installed on passenger lines in Great Britain were put in at Grateley on the South Western Railway by Mr. O'Donnell who was, also, one of the earliest advocates of concentration sidings and hump shunting, and established the big installation at Wath on the Great Central Railway.

Extensive contracts were carried out under his direction on the Great Central Railway, the London and Southwestern Railway, and the Central Argentine Railway, etc. At Victoria Station (South Eastern and Chatham Railway) the latest form of signaling—three-position, all electric interlocking—was finished at the time of his death.

Mr. O'Donnell was the inventor of a large number of appliances for railway signaling and kindred subjects, and was always well to the

* Memoir prepared by J. Groom, Esq.

fore in civil and mechanical engineering developments in connection with railways.

He was a Member of the following: Institution of Civil Engineers (Great Britain), Institution of Mechanical Engineers, Canadian Society of Civil Engineers, Railway Signal Association, and the Institution of Railway Signal Engineers. He was a Fellow of the Chartered Institute of Patent Agents.

At the time of his death Mr. O'Donnell held, among others, the following positions: Managing Director of the Superheater Corporation, Limited; Managing Director of the British Power Railway Signal Company, Limited; and Managing Director of the Economical Boiler Washing Company, Limited.

Mr. O'Donnell was elected a Member of the American Society of Civil Engineers on July 5th, 1893.

JOHN CHRISTIAN OSTRUP, M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1919.

John Christian Ostrup was born on an estate near Frederickshaven, Denmark, on August 31st, 1867. He received his early education at a private school in Frederickshaven and his principal technical training, including thorough military instruction, at the Royal Danish Military Academy, from which institution he was graduated in 1888. He remained in the Danish Army as Lieutenant for two or three years.

In 1891 Mr. Ostrup came to the United States and immediately supplemented his technical education by night work at an engineering school in Chicago, Ill., mainly for the purpose of learning English technical terms. He was graduated from that school in 1892. After obtaining his degree there, he engaged in various enterprises in different parts of the United States, beginning with the design of the Machinery Building for the Chicago World's Fair, and afterward including much work with the writer, first, on the Northwestern Elevated, Union Loop Elevated, and Lake Street Elevated Railroads, of Chicago, and, later, on the Elevated Railroad of Boston, Mass. On these vast enterprises he did effective and important work and held responsible positions. At first, his chief occupation was in the field, but, later, he did considerable structural steel designing, especially for the Boston Elevated, including the entire design of its great terminal building. On the completion of the latter, Mr. Ostrup spent a few years in private practice, mainly in the designing of power houses, industrial plants, etc., and the sheds to contain them; he then entered the employ of the American Bridge Company, where he remained for several years. While with that organization, he detailed, among other structures, the Manhattan Bridge.

From 1907 to 1911 he was Professor of Structural Engineering at Stevens Institute of Technology, Hoboken, N. J., giving there a most successful and popular course, notwithstanding the fact that he was handling a civil engineering department in a mechanical engineering school. It was at that time that he wrote and issued his excellent book, known to the Profession as "Ostrup's Specifications," which deals with methods of modern engineering. It has been widely used in military academies throughout the country, including West Point.

After four years' service at teaching, Mr. Ostrup resigned his position as Professor at Stevens Institute to take that of Chief Engineer to the Texas Company, with headquarters in New York City; and he handled with phenomenal success all the important engineering problems of the vast and diversified business of that company until it appeared probable that the United States would enter the World War, when he at once tendered his services to the Government of the land of

* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

his adoption. Having been thoroughly trained in military engineering in Denmark, his offer was gladly accepted, and he was given the rank of Major early in January, 1917. He sailed for France that year with the first detachment of the American Expeditionary Force, following General Pershing, to whose staff he had been assigned as an expert on engineering. During the war Maj. Ostrup saw continuous service at various points along the entire western front, his special duties being the supervision of the building of trench fortifications and bridges, where his knowledge of concrete construction proved of great value. In fact, he showed such unusual ability that, after a year at the front, he was returned to the United States to instruct new officers in the principles and practice of modern warfare. He continued his work as Instructor until after the declaration of the Armistice, when he tendered his resignation and asked for its speedy acceptance. He was urged to remain permanently in the United States Army, but refused, owing to ill health caused by over-work and an attack of influenza contracted while in France and from which he had never fully recovered. As soon as he was released from the Army, he returned to his old position with the Texas Company and resumed his former work.

On February 19th, 1919, Maj. Ostrup took lunch with the writer, and spent the better part of two hours in discussing his experiences abroad. It appears that, although often under fire, he was never wounded, although once his clothing was torn by a piece of shrapnel. His description of his work "over there" and of life in the trenches was intensely interesting, especially as he and the writer had been close friends for more than a quarter of a century. Among other things he mentioned the fact that, owing to the great amount of business done by the Texas Company during the war, his stock therein had brought such big returns as to render him independent for the rest of his life, and he was seriously considering retiring soon from active practice—but, alas! it was not to be; for that very evening he was stricken with influenza, which soon turned into pneumonia, to which he succumbed a week later.

Maj. Ostrup was a man of strong personality, an accomplished linguist, speaking several languages fluently and well, and having a reading knowledge of others, a deep student of applied science as well as of economics and the humanities, a good technical writer, an engineer of high attainments, a military officer of courage and of unusual ability, an accomplished musician, a true and loyal friend, and a thorough gentleman. In his premature death the Engineering Profession has sustained a real loss, and many members thereof have suffered a bereavement which will long be remembered.

Maj. Ostrup was elected an Associate Member of the American Society of Civil Engineers on March 4th, 1896, and a Member on March 1st, 1899.

HENRY KINDER OWENS, M. Am. Soc. C. E.*

DIED MAY 15TH, 1919.

Henry Kinder Owens, the son of William D. and Mary J. (Milham) Owens, was born in Wilmington, Del., on October 9th, 1858. Early in his life, the family removed to Missouri where the boy acquired his basic professional education, obtaining a C. E. degree in 1880.

Following his graduation, Mr. Owens engaged in railroad location and construction work in Missouri, Arkansas, and New Mexico. He went to the State (then Territory) of Washington in 1886 and began to interest himself in irrigation projects, in conjunction with railroad development, in Eastern Washington and Oregon. In 1887, he located in Seattle and was engaged as Construction Engineer on the Seattle, Lake Shore, and Eastern Railway, now a part of the Northern Pacific System.

In 1888, Mr. Owens entered into partnership with the late John G. Scurry, under the firm name of Scurry and Owens, and from 1888 to 1890, inclusive, this firm did the engineering work of the City of Seattle, including the replatting and regrading of the business district following the disastrous fire of June 6th, 1889. The firm also participated in the original plans of the present water supply and sewerage systems of Seattle, in collaboration with the late Benezette Williams, Consulting Engineer, of Chicago, Ill. The firm dissolved in 1892, and Mr. Owens planned and constructed water-works plants for some of the smaller communities in the State of Washington.

From 1893 to 1897, he held a special assignment with the Land Department of the Northern Pacific Railway Company, as Irrigation Engineer, in which capacity he made examinations, preliminary surveys, and reports, with estimates of construction costs, for projects embracing a large part of the irrigable lands in the land grant of that company, in Oregon, Washington, Idaho, and Montana. In 1898, he became Consulting Engineer for the Northern Pacific Irrigation and Land Company, in the execution of a project watering 110 000 acres in Yakima Valley. This project has been in successful operation for nearly 20 years.

Thereafter, he was Chief Engineer of the Hanford Irrigation and Power Company, as well as the Pasco Power and Water Company, and works of magnitude were built under his direction.

From 1908 to his death, Mr. Owens maintained an office in Seattle, as Consulting Engineer, in which period he successfully carried out many undertakings in hydraulic and railroad work, as well as in mining, sewerage, irrigation, and power plants, including among

* Memoir prepared by A. V. Bouillon, M. Am. Soc. C. E.

others, notably, the 20 000-kw., hydro-electric plant known as the Puntledge River Plant, built in 1911-13 for the Canadian Collieries, Limited, one of the McKenzie and Mann corporations in British Columbia. He also developed preliminary plans and estimates for numerous hydro-electric projects in Western Washington, and was interested in the development of the steel and nitrate industries in conjunction with hydro-electric power for this district. During part of this period (1911-17), he was associated with A. V. Bouillon, M. Am. Soc. C. E., also a Consulting Engineer of Seattle.

Mr. Owens possessed a genial character which was a source of pleasure to his associates. Always unassuming in manner, he never claimed the professional distinction to which his ability and accomplishments entitled him.

He was married on May 5th, 1887, at North Yakima, Wash., to Miss Alice M. Cock, who, with two sons, Henry Victor Owens, of Seattle, Wash., and Lieut. Theodore Owens, A. C., U. S. N., Naval Air Station, Hampton Roads, Va., survives him. He is also survived by his mother Mrs. Mary J. Owens, and a sister, Mrs. Mary J. Hansberger, both of Sedalia, Mo., and two brothers, Will M. and Fred J. Owens, of Atlanta, Ga.

Mr. Owens was elected a Member of the American Society of Civil Engineers on March 6th, 1889.

LOGAN WALLER PAGE, M. Am. Soc. C. E.*

DIED DECEMBER 9TH, 1918.

Logan Waller Page, born at Richmond, Va., on January 10th, 1870, came from a family distinguished in the public affairs of his State from the early days of the Jamestown Colony. He was a cousin of the late Walter Hines Page, recently Ambassador to Great Britain, and of Thomas Nelson Page, Ambassador to Italy, and a near relative of Dr. Thomas Walker Page, Chairman of the United States Tariff Commission. His technical education was obtained at the Virginia Polytechnic Institute and the Lawrence Scientific School of Harvard University. The late Nathaniel S. Shaler, his uncle, was Professor of Geology at Harvard University, and being a man who delighted in utilizing scientific knowledge for practical purposes, he became a strong advocate of a thorough scientific investigation of road-building materials and their correct use in highway construction. To put his ideas into effect, Professor Shaler, as Chairman of the Massachusetts Highway Commission, made Mr. Page its Geologist and Testing Engineer, in 1893.

As the Geologist of the Massachusetts Highway Commission, Mr. Page made the first extensive scientific investigation of road-building materials conducted in America. Although previous investigators had furnished information of value to road builders, it had been acquired incidentally to other geologic work. Mr. Page also inaugurated the petrographic study of these road-building materials, for the purpose of determining the relation between the composition of rocks and their behavior under traffic. In these early studies he demonstrated his possession of that rare scientific patience essential in order to discover the reasons for many of the empirical practices of engineering, which open a way for the rational improvement of older methods. The first comprehensive report on the constituents of road-building rocks which was printed in the United States was prepared by him in 1898. Some years later, in conjunction with Dr. E. C. E. Lord, he elaborated the programme of petrographic study of road-building materials which has been continued under his direction for many years, and about two years ago, with Dr. G. E. Ladd, he inaugurated a study of those features of economic geology which must be investigated scientifically in order to put road-stone quarrying on a satisfactory basis, both scientifically and practically.

The physical properties of rocks which affect their usefulness for road-building had been investigated by the French Bridge and Highway

* Memoir prepared by John M. Goodell, Assoc. Am. Soc. C. E., H. K. Bishop, M. Am. Soc. C. E., and T. Warren Allen, M. Am. Soc. C. E.

Corps for about twenty years when the Massachusetts Highway Commission undertook the same work. The French engineers had developed the Deval machine for testing the resistance to wear of rocks, the Dorry machine for testing hardness, and a machine for testing resistance to impact. Specimens of such apparatus had been used to some extent in the Massachusetts Institute of Technology, but it was not until Mr. Page introduced and improved the machines in the laboratory of the Massachusetts Highway Commission that physical tests of road-building rocks were regularly conducted as part of the routine work of a highway department in America. This step was taken in 1893. The Deval apparatus has remained in use with little modification since it was first introduced in France. The Dorry test is no longer used to any extent. Tests of the binding property of rock dust and of the toughness of rock were developed by Mr. Page for the Massachusetts Laboratory and subsequently improved under his direction in his laboratory at Washington. It might be added that the principles of the Page impact machine have recently been found useful in testing explosives. In these days, when such tests are carried on regularly in many laboratories, the part played by Mr. Page in introducing and improving them is likely to be forgotten.

In 1894 the United States Department of Agriculture had organized, under a $10 000 appropriation, an Office of Road Inquiry, mainly devoted to statistical and economic investigations. About 1899 Dr. Shaler discussed, with Secretary Wilson of the Department, the importance of conducting on a national scale the scientific study of road-building materials inaugurated by the Massachusetts Highway Commission. As the researches recommended by Dr. Shaler required laboratory facilities not possessed by the Office of Road Inquiry, Mr. Page, in 1900, was invited to become Chief of a Division of Tests in the Bureau of Chemistry. Here, with the assistance of Dr. A. S. Cushman, he inaugurated the series of scientific investigations which have since won international fame for the laboratories under his direction. These scientific investigations were later developed and extended under his direction, so that to-day they cover, not only the materials of road-building, but also the allied fields of research in which greater knowledge is enabling highway engineers to use more economically the money spent under their direction.

In the course of his studies of road materials, first in Massachusetts and later in Washington, Mr. Page acquired an intimate knowledge of road-building. After moving to Washington his engineering knowledge and firm grasp of the essential features of highway economics gradually won such recognition that Secretary Wilson consolidated all branches of the highway work under his direction in the Office of Public Roads, with Mr. Page as Director. This was in 1905, and, from a very small beginning, the work of the Office of Public Roads has developed and

extended until, at his death, his bureau was co-operating in road construction with every State.

In the early days of his directorship, Mr. Page found it necessary to visit many parts of the country, and to discuss the desirability and proper character of road improvement programmes for many different conditions. The importance of this work impressed him so strongly that in 1910 he secured the co-operation of a number of prominent men, among them the late Alfred Noble, Past-President, Am. Soc. C. E., in founding the American Highway Association, which conducted an active educational campaign for rational road improvements as long as such work was necessary. The organization of this Association and the useful work it did are proofs of the resourcefulness shown by Mr. Page in carrying out his plans for the betterment of the country's highways.

For many years he was engaged in two distinct lines of road work, educational and research. Under his direction, lectures on road topics were given in all sections of the country, good-road demonstration trains were run on many railroads, and numerous bulletins on the value, construction, and maintenance of roads were published. The value of this pioneer work done by the Department of Agriculture can be seen in the practically universal recognition of the importance of good roads. Where the conditions warranted it, special advice was given to communities contemplating the inauguration of highway betterments, in order that these scattered examples of good administration and good engineering would serve as models for other communities. In some cases actual demonstrations of the best methods of construction and maintenance were made. Later, when the drainage, irrigation, and rural engineering work of the Department of Agriculture were put under his direction, in 1915, the same general system was followed with these branches. The organization at this time became the Office of Public Roads and Rural Engineering, and later simply the Bureau of Public Roads.

The research work conducted under Mr. Page's direction embraced, not only laboratory investigations, but also service tests with experimental roads. The latter have been built in many parts of the country, with numerous types of construction. The service to which they are subjected is determined accurately and frequently, and in this way the facts regarding disputed features and materials of road-building have been ascertained. Then, by comparing laboratory and service results, the two are correlated, and furnish a guide for further investigations. This nation-wide research is now having one interesting result in a decided tendency toward the standardization of specifications for road materials. The work not only embraced the construction of roads, but also included their maintenance, a subject always uppermost in Mr. Page's mind, on account of its great financial importance. It

should be added here, that, on the advent of the automobile and the early indications of its destructive influence on roads, Mr. Page made a very careful investigation of the nature of this wear, and published both popular articles and technical papers advocating more durable forms of construction in order to meet the new conditions of travel.

Mr. Page's active participation in Federal road-building began in 1912, when Congress appropriated $500 000 for improving certain post roads. In 1916 Congress voted to give $75 000 000 to help road-building in the various States on a co-operative basis, the States providing at least an equal amount. The administration of this work was placed with the Secretary of Agriculture, who delegated the direction of it to Mr. Page. It was a new venture, and the early proceedings were complicated by delays incident to the necessity of obtaining legal approval of many of the administrative methods, and legal interpretation of various parts of the law. These delays caused some criticism, but, as time passed, the wisdom of proceeding in strict conformity with the law became apparent, and the great National value of Federal aid in road-building became clear. At the time this memoir is prepared, Congress has before it a bill to increase substantially the Federal appropriations for such work.

One of the most conspicuous examples of Mr. Page's foresight in matters affecting road work occurred after the United States took up arms. The congestion of the railroads led to the first drastic action restricting road-building, namely, an order forbidding the use of open-top cars for hauling road materials. Mr. Page obtained a modification of that order. Somewhat later, an order was issued forbidding the use of bituminous materials for road work. Mr. Page had this modified, after considerable difficulty. At his request, the Secretary of Agriculture then invited the Government departments and agencies affected in any way by road work to join with him in establishing a United States Highways Council to exercise control over the road work of the country during the period of the war. As a result of this invitation, the Highways Council was formed, and the allocation of available road materials and their transportation, and the approval of capital issues for road work were passed upon by a single body, which was able to assist road work, so far as the war programme made this possible. Mr. Page was the Chairman of the United States Highways Council, and the Bureau of Public Roads furnished most of the facilities and personnel for its work.

Before the administrative duties of the Bureau of Public Roads became so heavy as to demand most of his time, Mr. Page wrote numerous papers on subjects connected with road building, one of which, on oil-concrete, was contributed to this Society.* He was the

* "Some of the Properties of Oil-Mixed Portland Cement Mortar and Concrete", *Transactions*, Am. Soc. C. E., Vol. LXXIV (1911), p. 225.

author of "Roads, Paths, and Bridges," a non-technical discussion which had a wide circulation.

He was a man of strong, forceful personality, dominated by an unusually constructive imagination. He could develop a plan mentally and carry its main features in his mind without any apparent effort or gradual loss of clearness. This was a great gift for a Federal bureau chief, as it enabled him to pick up from time to time the materials and men needed to carry out an undertaking which would only become practicable when the resources for it were available. It was this planning far ahead which led many to regard him as idealistic; it was not dreaming, however, but an unusual faculty for preparing to meet the demands on his bureau, which he knew would be made later. It was this characteristic, furthermore, which made the research work under his direction very largely pioneer work, and gave the laboratories of the Bureau of Public Roads the unique place they occupy to-day.

Mr. Page was elected a Member of the American Society of Civil Engineers on July 1st, 1909.

MYRON HALL PECK, M. Am. Soc. C. E.*

DIED OCTOBER 9TH, 1918.

Myron Hall Peck was born at Racine, Wis., on October 13th, 1876. He was the son of Dr. Albert P. and Celia Flagg Peck. His father served in the Civil War, although he was only 17 at the time of his enlistment. In 1880, the family moved to Shantung Province, China, where Dr. Peck was a medical missionary under the American Board of Commissioners for Foreign Missions—the Congregational organization—for more than 20 years, and Myron Hall Peck's boyhood was spent in China and in California. He was graduated from the College of Civil Engineering at the University of California with the Class of 1897, with the degree of B. S.

After graduation, he engaged in civil engineering and mining work in various parts of California until 1902, when he returned to China to become Professor of Civil Engineering in the Chinese Imperial University in Shansi and, later, at the Imperial Pei Yang University at Tientsin, which position he held until he came back to the United States in 1911.

Mr. Peck's first work after his return was for the War Department, on the improvement of the bar at the mouth of the Columbia River. Following this, he went to San Francisco, where he opened engineering offices, which he maintained most of the time until 1916. During part of this time, he was engaged as Engineer in Charge of the improvement of the channel at Mare Island Navy Yard, for the Standard American Dredging Company, the contractors doing the work.

In 1916, he enlisted in Company A of the Engineer Corps of the California National Guard, and served as Second Lieutenant during the Mexican Border troubles, being stationed most of the time at Nogales, Ariz. By special arrangement, he was mustered out of the service in December, 1916, at the request of the Citizens' Committee of Vallejo, Cal., in order that he might prepare a report on channel conditions and other factors which might influence the possible enlargement of the Mare Island Navy Yard, the Committee recognizing that his experience in connection with the improvement of the channel made him particularly fitted for this work. Later, he went to Washington on behalf of the Committee.

Soon after the Engineer Officers' Reserve Corps was created, Mr. Peck applied for a commission, and was assigned for training to Camp Lee, Va., early in 1917. He was commissioned a Captain and spent several months in training troops at Camp Lee. In January, 1918, he sailed for France. Then followed a brief period of further work back of the lines, but the last few months of his life were mainly spent

* Memoir prepared by A. T. Parsons, Assoc. M. Am. Soc. C. E.

at the front with the Second Division, commanded by Major-General John A. Lejeune. Captain Peck was at Chateau Thierry, St. Mihiel, and in many less known engagements. During a large part of the time he acted as Battalion Commander, his untimely death doubtless preventing his promotion to the rank of Major.

On October 9th, just after he had led his battalion through a particularly brilliant action near St. Etienne-a-Armes, which won for him the posthumous honor of the Distinguished Service Cross, Captain Peck was struck by a fragment of shell and killed instantly.

No better idea of the stuff of which Captain Peck was made, as well as the circumstances of his death, can be given than by quoting from the following letters, the first being from the First Sergeant of his Company:

"At 8 A. M., October 9th, we were being attacked and the 'boche' put over a murderous barrage. Captain Peck was standing in the cemetery, which we occupied and regarded as our strong point, directing his men. He seemed oblivious to all danger and fell in the performance of his duty. * * *

"This regiment has suffered a severe loss, as the late Captain was a model soldier and gentleman in every respect. As my company commander, I had been on several reconnaissance and other details with him, and I learned to admire his soldierly ability and gentlemanly qualities. In fact, I felt perfectly safe in following Captain Peck anywhere, and I lost a close friend when death called him away.

"JOHN D. DRURY,
"First Sergeant, Company E, Second Engineers."

The second extract is from a letter from the Colonel of his Regiment:

"Captain Peck was killed in action during particularly difficult fighting. His death came directly as a result of his fearless exposure of himself in doing his duty. He was beloved by his men for his personality, his qualities of leadership, and his apparent fearlessness.

"His body was immediately recovered by his comrades, and as carefully buried as was possible under existing circumstances. Later, it was removed to a French cemetery. * * * The headstone was carved by one of his men, who did all the carving with a pocket-knife. It was a labor of love. * * *

"His efficiency was becoming increasingly evident, and he would soon have been made a Major. In fact, he had been actually commanding the battalion for two months.

"W. A. MITCHELL,
"Colonel, Second Engineers."

Although these letters do justice to the gallant soldier who has gone from us, they do not give a complete idea of the capable engineer and the true friend it was a privilege to know.

Captain Peck was a charter member of the Engineers' Club of San Francisco, having served as Vice-President during the year following its founding in 1912.

In 1905, Captain Peck was married to Miss Mary Moore, who, with two children, Gladys Logan and Albert Carlton Peck, survives him. He is also survived by his parents, Dr. and Mrs. A. P. Peck, of Tientsin, China, by two brothers, Willys R. Peck, U. S. Consul at Tsing-Tao, China, and Llewellyn B. Peck, of San Francisco, Cal., Second Lieutenant in the Aviation Section, Signal Reserve Corps, U. S. Army, and one sister, Mrs. Thomas T. Read, of Montclair, N. J.

Captain Peck was elected an Associate Member of the American Society of Civil Engineers on January 5th, 1909, and a Member on March 14th, 1916.

JOHN HOWE PEYTON, M. Am. Soc. C. E.*

DIED SEPTEMBER 14TH, 1918.

John Howe Peyton was the fourth son of William Preston Peyton, of Montgomery County, Virginia, and Sarah Elizabeth (Mumford) Peyton of Richmond, Va. His paternal grandfather, Major Garnet Peyton, was in active service in the United States Army during the War of 1812. His maternal grandfather was a man of great literary ability, whose chief work was the translation of Homer's Iliad into blank verse. John Howe Peyton was born in Howard County, Missouri, on March 17th, 1864, his parents having thought to make their future home there, but returned to Virginia soon after the close of the Civil War. His family, like many others, suffered many hardships as a result of the war, but his father succeeded in giving each of his five sons a college education. In December, 1873, the family moved to Salem, Va., and he entered the Preparatory Department of Roanoke College, finishing his course in 1881.

At that time railroad construction and surveys were active, and Mr. Peyton joined one of the engineering parties of the Richmond and Louisville Railroad, a project which was then being pushed.

In 1883 he was Instrumentman for the Farmville and Powhattan Railroad, and from 1885 to 1888 he was Resident Engineer on construction for the Lynchburg and Durham Railroad, later holding a similar position with the Georgia, Carolina and Northern Railway. From 1889 to 1891 he was Assistant to the Chief Engineer of the Charleston, Clendenning and Sutton Railroad.

From 1892 to 1898 Mr. Peyton engaged in private practice as an Engineer and in contracting, having his office at Charleston, W. Va. In 1899 he joined the United States Army, and became a Sergeant in the Engineer Corps, and, later in the year, went to the Philippines as special representative of the Protestant Episcopal Church.

From 1900 to 1901 he was Resident Engineer of the Chesapeake and Ohio Railroad, and, later, was made Chief Engineer of the Great Eastern Railroad, serving in that capacity until 1902, when he entered the service of the Louisville and Nashville Railroad Company as Locating Engineer. His work for this company consisted of surveys and estimates for extensions, and for reducing grades on their existing lines. He at that time located the revision of the line and grade of the Henderson Division, between Edgefield Junction and Hygeia, where a 4% grade was replaced with a 0.7% grade, requiring very heavy work and a tunnel about 4 000 ft. long. He also located the Atlanta, Knoxville and Northern Railway, from Etowah, Tenn., to a junction

* Memoir prepared by Hunter McDonald, Past-President, Am. Soc. C. E.

with the Western and Atlantic Railroad, at Junta, Ga. He was placed in charge of the construction of these two lines, but was made Principal Locating Engineer for the company in 1904, and served in that capacity until 1907.

After the panic of 1907, Mr. Peyton left the service of the Louisville and Nashville Railroad Company, and accepted a position with the Union Pacific Railroad in the Northwest, making investigations in Montana, Washington, Oregon, and British Columbia. He returned to the service of the Louisville and Nashville Railroad Company in 1910 as Assistant to the President.

In March, 1912, he was made Chief Engineer of Construction, and carried the title of Assistant to the President and Chief Engineer of Construction, Louisville and Nashville Railroad Company. His duties consisted of locating and superintending the construction of a number of very heavy pieces of grade reduction and extension work, notably the Lewisburg and Northern, extending from Portland, Tenn., to Decatur, Ala., the revision of the South and North Alabama Railroad, between Decatur and Birmingham, the Lexington and Eastern Railroad, an extension from Covington, Ky., into the coal fields of Eastern Kentucky, and the Tuscaloosa Mineral Railway.

In January, 1914, Mr. Peyton was appointed General Manager and, in April following, President of the Nashville, Chattanooga and St. Louis Railway Company. He assumed the duties of these offices with preconceived ideas of the obligations of the railroads to the public, and entered at once into a vigorous campaign to put these ideas into effect.

He traveled over the lines of road on trains and in motor cars, studying location and grade-reduction problems, and, where his view from the train was circumscribed, he made many trips in automobiles through the mountains, studying the topography. He organized surveying parties, appointed a Superintendent of Construction, and personally directed the work. He revised the existing schemes for grade reduction between Sherwood, Tenn., and Paducah, Ky., by way of Nashville and Hollow Rock Junction.

During his term of office a considerable portion of this work was begun, some of it was executed, but the breaking out of the war with Germany compelled the stoppage of most of it. However, pending the completion of the grade-reduction plan, the tonnage of trains was greatly increased by the purchase of larger engines and the use of helping engines at points where they had not been utilized previously.

Mr. Peyton organized an Agricultural Department of the Railway, bought demonstration farms in a number of counties, established modern farming methods thereon, and, under the direction of a Special Agricultural Agent, these farms were instrumental in greatly improving

farming methods throughout the South. He made many trips over the road, speaking to citizens and at Farmers' Institute meetings.

On the assumption of control of the railroad by the Government, Mr. Peyton was called on to make several studies for the Regional Director of the Southern Region, and reported on the navigation scheme on the Mississippi River between St. Louis and New Orleans, and on the Black Warrior River from the coal fields of Alabama to Mobile and New Orleans. He was peculiarly fitted for this assignment, having previously made a study of inland navigation and written a book, entitled "The American Transportation Problem", which dealt particularly with comparisons between railway and inland transportation by water.

Before he assumed the Presidency of the Railway, the project of building the Paducah and Illinois Railroad, which included a bridge over the Ohio River at Metropolis, Ill., had taken shape, and work had begun, but, on the breaking out of the war, it was stopped. Mr. Peyton succeeded in re-organizing the work, and, through his efforts, it was brought to a successful conclusion, under the direction of C. H. Cartlidge, M. Am. Soc. C. E., Bridge Engineer of the Chicago, Burlington and Quincy Railroad, and Chief Engineer of the Paducah and Illinois Railroad, who, however, died before the bridge was entirely completed.

Mr. Peyton did not remain with the Nashville, Chattanooga and St. Louis Railway under Federal control, but retained the Presidency of the Corporation. Just prior to this change, however, he undertook, for the railway, the construction of the Old Hickory Powder Spur for the United States Government, which was completed in a remarkably short time. After the institution of Federal control of the railways, Mr. Peyton made application for a Commission in the Engineering Corps, desiring to enter the American Expeditionary Forces.

Mr. Peyton was a very devout member and lay-reader of the Protestant Episcopal Church. He was a Trustee of the University of the South, an Episcopal School, at Sewanee, Tenn., of George Peabody College for Teachers, at Nashville, Tenn., a Director in the Andrew Jackson Memorial Association, in the Monteagle Assembly and Summer School, and in the Southern Howard Association, and was active in all.

He was President of the Nashville Section of the Engineering Association of the South in 1916, was a Member of the American Railway Engineering Association, of the Tennessee Historical Society, and the Tennessee Academy of Science. He was also a Member of the Old Oak Club, the Rotary Club, the Hermitage Club, and the Commercial Club, in Nashville.

He was intensely public-spirited, religious, and charitable, devoting much of his time and money to these causes.

Mr. Peyton came of ancestors who were distinguished for their statesmanship and service to the State of Virginia and the Nation. He was very proud of Virginia, and took a deep interest in her welfare and in the welfare of the South. He was especially devoted to young men, and attached many of them to him through his kindness and consideration for their advancement. It was always a source of regret to him that his parents made their excursion to Missouri, and of joy that they returned to their beloved Virginia after so short a stay in the West.

Mr. Peyton was never married. His death occurred at his home on West End Avenue, in Nashville, Tenn., on the morning of September 14th, 1918. Two elder sisters, Misses Elizabeth M. and Sallie R., survive him, with their nephew, Wythe M. Peyton, who resides near Jennings, Va. There are also two elder brothers, William M. Peyton, of Martinville, Va., and Carlton P. Peyton, of Charleston, W. Va. His other nephew, the Rev. William P. Peyton, served with the American Expeditionary Forces in France.

Mr. Peyton was elected a Member of the American Society of Civil Engineers on June 1st, 1909.

ROBERT WINTHROP PRATT, M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1920.

Robert Winthrop Pratt was born on December 23d, 1876, in Brookline, Mass. He was descended from old New England families, his father—a descendant of John Alden—being Robert Winthrop Pratt, and his mother, Grace (Kellogg), Pratt.

Mr. Pratt's early education was obtained in the public schools of Boston, while his technical education was acquired at the Massachusetts Institute of Technology, from which he was graduated as a Civil Engineer in 1896.

After his graduation, Mr. Pratt began his practical engineering experience with the Boston and Albany Railroad. After about a year of railroad work, he left this field and started his career in sanitary engineering as Assistant Engineer with the Massachusetts State Board of Health. He remained with the State Board of Health for four years and then moved to Ohio, where he was engaged as Chief Engineer of the Ohio State Board of Health, holding this position from 1903 to 1911. During the early years of his work in Ohio, he also supervised for the Federal Government the gauging of stream flows in Ohio streams. This work was well performed and has proved to be of considerable value.

During one year of his connection with the Ohio Board of Health, namely 1909-10, Mr. Pratt was given a leave of absence to permit him to visit Havana as Consulting Engineer for the Cuban Government, in connection with the design of water supply and sewerage systems for several Cuban cities.

In October, 1911, Mr. Pratt left his position with the State Board of Health and moved to Cleveland, Ohio, where he became Special Sanitary Engineer for the latter city, and where he also began his private practice as Consulting Sanitary Engineer.

In connection with his work in Cleveland, he directed the tests and investigations which were conducted preliminary to designing the sewage disposal plants for the city. The early studies and general plans for the Cleveland Sewage Disposal System were also carried on under Mr. Pratt's general direction. During this same period, he also acted as Consulting Engineer for the Water Department, in connection with the design of the water filtration plant.

Mr. Pratt's professional life was an exceedingly busy one, and many places in Ohio and elsewhere are supplied with sewerage or water systems and with treatment plants built under his direction. He had also

* Memoir prepared by Robert Hoffmann and W. H. Dittoe, Members, Am. Soc. C. E.

prepared a number of important reports on water and sewerage projects, of which the preliminary studies and report on the Toledo, Ohio, sewerage problem and his investigation, tests, and report on the proposed filtration of the water supply of Detroit, Mich., may be mentioned as of special importance.

The World War found Mr. Pratt particularly busy in connection with engineering work for the Chillicothe Cantonment and with industrial housing developments.

He had prepared many printed reports in connection with his Board of Health work as well as descriptions of work designed by himself. He was joint author of the well-known book on "Sewage Disposal", by Messrs. L. P. Kinnicut, C.-E. A. Winslow, and R. Winthrop Pratt.

Mr. Pratt had a fine, pleasing personality and was a most agreeable associate, socially and in business. He had a faculty of taking a broad view of things which helped him wonderfully in matters coming to his attention. He had high ideals of engineering ethics which he practiced himself and expected others to observe. Through his untimely death, which was due to pneumonia and occurred on February 2d, 1920, after only a week's illness, the Engineering Profession loses a valued member and his closer acquaintances a true and faithful friend.

In June, 1903, he was married to Elizabeth Southwick, of New York City, who, with a family of two sons and two daughters, survives him.

Mr. Pratt was elected a Member of the American Society of Civil Engineers on June 6th, 1911.

HOMER PETER RITTER, M. Am. Soc. C. E.*

DIED APRIL 21ST, 1919.

Homer Peter Ritter was born in Cleveland, Ohio, on March 4th, 1855, and received his early education in the schools of his native city. On being graduated from the Cleveland High School, guided by an artistic temperament which was a marked characteristic through life, he entered the Cincinnati School of Design, but, after a short stay, withdrew from this institution and matriculated in the School of Mines, Columbia University, New York City. At Columbia he selected a special course in Civil Engineering, and, on its conclusion, in 1880, received an appointment in the service of the Mississippi River Commission and was at once assigned to the field. As the Commission was organized in 1879, Mr. Ritter was one of its first employees, and, in the five years between 1880 and 1885, was engaged as Assistant United States Engineer in the topographic and physical hydrographic operations on which were based the comprehensive plans for the regulation of the great river.

In 1885, Mr. Ritter was transferred to the U. S. Coast and Geodetic Survey, for employment in the physical hydrographic surveys and examinations, under Professor Henry Mitchell, that were a necessary antecedent for the improvements that have met the demand for the modernization of the waterways about New York City; and, later, on similar surveys in the waters adjacent to Boston and off the New England Coast; and on general hydrographic and topographic work of the Survey. In 1893 he was appointed American representative in a Canadian photo-topographic party that formed a unit in a group of American and Canadian parties which initiated the exploratory surveys and maps used by the Alaskan Boundary Tribunal in laying down the line of delimitation between the United States and British possessions between Mt. St. Elias and the Portland Canal.

In 1895, Capt. Ritter was appointed Assistant in the normal force of the Coast and Geodetic Survey, a designation of rank which was changed to that of Hydrographic and Geodetic Engineer in 1917. From 1898 to 1904 his work was confined entirely to the Pacific Coast, and included a large portion of the range of duties covered by the Survey. In 1898 his services included magnetic and astronomical operations in California and Nevada; from 1899 to 1903, in command of the U. S. Coast and Geodetic Survey steamer *Taku*, he was employed on triangulation, hydrography, and topography in Controller Bay, Prince William Sound, and about Kodiak Island; and in 1904 he commanded the U. S. Coast and Geodetic Survey steamer *McArthur*

* Memoir prepared by R. I. Faris, M. Am. Soc. C. E.

engaged in special surveys about Kiska, one of the extreme western islands of the Aleutian Group.

In 1904, Capt. Ritter, on nomination by President Roosevelt, was confirmed by the Senate as a member of the Mississippi River Commission, thereby renewing his connection with the body which he had entered at the beginning of his professional career, and in which he was destined to render his last service, for it was from a regular trip of inspection of the Commission that he returned incapacitated by the illness which caused his death within four days after his return to his home in Washington, D. C.

In 1904, Capt. Ritter was transferred from duty on the Pacific Coast to the Coast and Geodetic Survey Office in Washington, D. C., and the remainder of his services were rendered on the Atlantic Coast, and, in the interval between the beginning of 1905 and his death on April 21st, 1919, these were of a greatly varied and highly valuable character. Besides his active employment for 15 years as a member of the Mississippi River Commission, he was engaged as a member on the 14-ft. Waterway Board; completed a thorough physical hydrographic survey of the Potomac River, in connection with the determination of the causes and correction of its pollution; repeatedly was assigned to determine and locate speed courses for the Navy; make extensive resurveys for chart revision in Chesapeake Bay, Long Island Sound, and along the coast of Massachusetts; and conducted an extensive triangulation in North Carolina. His last assignment to duty in the Coast and Geodetic Survey was to the position of Inspector in Charge of the U. S. Coast and Geodetic Survey Field Station at Boston, Mass., and during this incumbency, which continued to the day of his death, he was conducting revision surveys along the Massachusetts Coast.

Capt. Ritter was a man of unostentatious but admirable qualities, and especially distinguished by a rare type of humor which was blended with a singular knowledge of humanity and a depth of sympathy that seemed unfathomable. It is related of him that when but little more than a youth, during one of the great floods in the Lower Mississippi, in the face of a danger that forbade others to adventure it, he went, alone, in a dugout, to the rescue of an unfortunate negro who, marooned on the roof of a cabin, was being hurried to destruction by the raging river, and brought the man to safety. At another time he and one companion, during an outbreak of fever, resisting the fear that drove off their other well companions in panic, abided with those stricken down by the disease until all danger from contagion had passed and medical assistance could be brought to them.

At a meeting of the Mississippi River Commission on April 23d, 1919, the following resolution on the death of Capt. Ritter was presented by J. A. Ockerson, Past-President, Am. Soc. C. E., and carried:

"An all wise Providence has suddenly called from our midst our friend and associate Homer P. Ritter, and we desire to spread upon the records of the Mississippi River Commission this statement of the deep sorrow which has fallen upon us without warning and expressions of our sympathy for his family with whom we mourn his loss.

"He entered the work of the Commission in 1879 and rendered valuable aid in the physical investigations of the Mississippi River. After serving six years in that capacity he was called to the service of the Coast and Geodetic Survey where he was still a prominent member of its corps of experts when he came to his untimely end.

"He was appointed a member of the Mississippi River Commission in 1904, and his conscientious devotion to duty was such that although ill he insisted on accompanying the Commission on its inspection trip leaving St. Louis on April 15, 1919, but on reaching Memphis, April 17, illness compelled his return to his home in Washington, and he bade us a sad farewell.

"He will linger with us in fond remembrance for his kindly, genial friendship and his good qualities as a valuable Member of the Commission.

"As a matter of respect to his memory, be it resolved that the flag on the steamer *Mississippi* be carried at half mast during the present session of the Commission."

Capt. Ritter was married on April 12th, 1912, to Miss Katherine R. Wagner, of Piqua, Ohio, and, besides his widow, is survived by two brothers and two sisters. He had been a member of the Cosmos Club of Washington since 1892.

Capt. Ritter was elected a Member of the American Society of Civil Engineers on November 3d, 1915.

ALBERT ALONZO ROBINSON, M. Am. Soc. C. E.*

DIED OCTOBER 7TH, 1918.

Albert Alonzo Robinson was born in the little Village of South Reading, Vt., on October 21st, 1844. He was a son of Ebenezer Robinson, Jr., and Adelaide (Williams) Robinson. For generations his ancestors had been New Englanders. His grandfather, Ebenezer Robinson, fought in the Revolutionary War. His father, Ebenezer, Jr., was a country school teacher, farmer, and carpenter. He was a mechanic of no mean ability, and the talent he possessed was passed on to his three sons, all of whom have been more than successful in the particular line of engineering in which they embarked. He died in 1848, leaving his wife with a family of four children. Five years after the death of her husband, she married Mr. Albert Childs, and the family moved West, where Mr. Robinson was to stamp the mark of his genius on the country by constructing a great railway system. Although he was held down by the necessity of work and the hardships usually faced by the pioneer Westerners, he received a college education. He attended the Academy at Wilton, Wis., after finishing the common school course, and, later, entered the University of Michigan, from which he received the degrees of Bachelor of Science and Civil Engineer, in 1869. In 1871 his Alma Mater conferred on him the degree of Master of Science, and, in 1900, further honored him with the degree of Doctor of Laws.

Relative to Mr. Robinson's early life: From childhood until he reached his majority he was employed in farm labor, except for a short period when he worked as a clerk in a grocery and dry goods store. While he was a student at the University of Michigan, he was occupied for about five months every year as an Assistant Engineer on the United States Lake Survey in astronomical and field work and in the triangulation of the Great Lakes.

His railroad work began in 1869, when he entered the service of the St. Joseph and Denver City Railroad. Mr. Robinson was actively engaged in work with the railroads of the West for a period of 37 years, and rose, by successive promotions, to be Vice-President of the Santa Fé and President of the Mexican Central Railway. It was in April, 1871, that Mr. Robinson entered the service of the Santa Fé, which road was then scarcely 100 miles long. When he left, 22 years later, it had grown into a system of more than 9 000 miles, more than half of which had been built by him.

Mr. Robinson's first work with the Santa Fé was on surveys and construction, between Topeka and Atchison. He was made Chief

* Memoir prepared by J. M. Meade, Esq., Special Engineer, Atchison, Topeka and Santa Fé Railroad, Topeka, Kans.

Engineer in 1873, and, later, held the following positions, in the succession in which they are given: Division Superintendent; Superintendent of Bridges, Buildings, and Water Service; Assistant General Superintendent; General Manager; Second Vice-President; Vice-President and General Manager.

In 1893 he resigned to become President of the Mexican Central Railroad, which position he held until his retirement in 1906, because of his advanced age and the state of his health. While he was President of the Mexican Central, his office was in Boston, Mass. He proved himself a very able administrator, as well as a great engineer, and developed that road into a great railway system. When he went to the Mexican Central, the stock of that company was selling at 5 cents a share and the bonds at 43 cents a share. When he retired, the stock was selling at 50 cents and bonds at 80 cents.

Having built between 5 000 and 6 000 miles of railroad, Mr. Robinson was known as one of the world's greatest civil engineers. The present lines of the Santa Fé Railroad were all constructed under his direction. Coming to the Santa Fé in 1871, he laid out hundreds of miles of that fast-growing railroad. As Chief Engineer, he outlined and executed the vast building operations, conceived and effected by Messrs. C. K. Holliday and W. B. Strong. As General Manager, it was his keen judgment that drew hundreds of men to the Santa Fé, and made it the leader of the railroads of the country. He was a leader who was loyal to his men; although twenty years have passed since he was connected with the Santa Fé, his name will go down in the history of that System, as an honest, loyal, staunch friend of his fellow workers.

Mr. Robinson was married in 1869 to Julia Caroline Burdick, of Edgerton, Wis., who died in August, 1881, leaving a daughter, Metta Burdick. On September 2d, 1885, he was married to Mrs. Frances Williams, a sister of his first wife, who, with his daughter and a granddaughter, survives him.

Mr. Robinson was one of Topeka's best known and respected citizens. His residence in that city, since 1875, after a period of nearly 40 years of railroad activity, speaks eloquently of his faith in the city's future and his high esteem of its citizenship. He had always taken a prominent part in local as well as National affairs, and was Past-President of the Topeka Chamber of Commerce, as well as a prominent member of many other local and civic organizations. In politics he was a Republican, and was always connected with the better things associated with the affairs of the city and country.

In 1876 he erected one of the grand residences of Topeka, which is known as one of the beauty spots of Kansas. His home contains one of the greatest collections of art in the West.

Mr. Robinson's death occurred on October 7th, 1918, after months of sickness, which had caused most of his close friends and relatives to lose hope of his recovery.

He was a member of many famous and exclusive civil, technical and social societies, among which were the Topeka Club, the Country Club, the Chicago Club of Chicago, Ill., the Lawyers Club of New York City, and the Jockey and University Clubs of the City of Mexico.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on May 5th, 1880.

JOHN WARREN SACKETT, M. Am. Soc. C. E.*

DIED JULY 3d, 1918.

John Warren Sackett was born at Rantoul, Ill., on December 13th, 1860. He received his elementary education at that place, and when he was 14 years old, he had been graduated from the High School and had passed the entrance examinations to the University of Illinois. He was prohibited from entering at that time, however, by reason of the age limit—15 years—in force there. In 1876 he became a student in the Civil Engineering Department of the University where he remained for two years. He also took manual training in carpentry and machine work.

In 1877 and 1878, Mr. Sackett taught school, with the intention of completing his college course, but his father, who had moved to Florida and was engaged in contracting and building, required his services in his business. In this connection Mr. Sackett made architectural drawings and superintended construction work, as well as operating a saw-mill. In the meantime, he continued his engineering studies under J. Francis LeBaron, M. Am. Soc. C. E., by whom he was engaged in land and railroad surveying and in the operation of a short line of railroad, as well as on Government surveys of various canals, rivers, and harbors in Florida.

On May 16th, 1881, Mr. Sackett was appointed an Assistant Engineer in the United States Engineer Corps, by the late Col. Q. A. Gillmore, Corps of Engineers, U. S. A., M. Am. Soc. C. E., on the recommendation of Mr. LeBaron, U. S. Assistant Engineer, under whom he was serving, and was identified with that organization for more than thirty-five years, as follows:

May and June, 1881, he served as Recorder in the party of Mr. Le Baron, under whose immediate direction he remained, until 1886, on the survey for a canal route from the head-waters of the St. Johns River to Charlotte Harbor, Florida; December, 1881, and January, 1882, he was engaged as Inspector on jetty work at Cumberland Sound, Georgia, and Florida; about this time he entered a competitive examination for appointment to West Point Military Academy and had been recommended for entrance, but owing to political reasons, the appointment was not granted; until the fall of 1883, he was Inspector of jetty work at the entrance to St. Johns River, Florida; until the spring of 1884, he was engaged as Instrumentman on a survey of the Savannah River; in 1885, he made a survey and plans for a Marginal Railroad in Jacksonville, Fla., under the direction of Mr. LeBaron, Chief Engineer, and surveyed and platted several town sites; he also served,

* Memoir prepared from information furnished by J. Francis LeBaron, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

until May, 1885, as Chief Draftsman in the Surveyor-General's Office at Tallahassee, Fla.; until February, 1886, he was Superintendent and Draftsman, under Capt. William T. Rossell, Corps of Engineers, U. S. A., on dredging and rock excavation at Tampa and Cedar Keys, Fla., and in the spring of the same year, he served as Assistant, under Capt., now Maj.-Gen., William M. Black, Corps of Engineers, U. S. A., M. Am. Soc. C. E., on the survey at the entrance to St. Johns River, Florida; and from that time until 1888, he was engaged as Assistant Engineer in charge of the survey at the northwest entrance at Key West Harbor, Florida, and of a survey of the Caloosahatchee River.

In 1888, Mr. Sackett was made Principal Assistant Engineer of the Florida District, and given general supervision of the details of the work in that District under the direction of the District Engineer Officer, continuing in that position until his sudden death on July 3d, 1918, his title having been changed in 1917 to that of Chief Assistant Engineer of the Florida District of the U. S. Engineer Department. His work as Principal and Chief Assistant Engineer of the District consisted in preparing plans, projects, estimates, reports, etc., for submission to the Chief of Engineers, as well as the detailed design construction, operation, maintenance, etc., of all the floating plant, such as hydraulic dredges, steam and power plants, hyacinth elevators, etc. In connection with this work, he had introduced many novel improvements in successful operation, on several of which patents have been issued.

While a student at the University of Illinois in 1876-77, Mr. Sackett had received some military training, and, in 1895, he was commissioned as Captain of Co. G, 1st Florida Infantry, known as the St. Augustine Rifles. When war with Spain was declared in 1898, Capt. Sackett was promoted to be Major in the 1st Florida U. S. Volunteer Infantry, but was afterward detached from that regiment and placed in command of a Provisional Engineer Battalion formed by details from Florida and Illinois troops. After training for about six weeks at Port Tampa, Fla., Maj. Sackett left there with the Battalion and arrived at Santiago, Cuba, the day following the surrender of the Spanish Forces. After some construction work at Guantanamo Harbor, the Battalion joined the Porto Rican Expedition and led the landing party at Guanico on July 25th, 1898, under fire, with Maj. Sackett in command of the right wing. For this action and for soldierly qualities displayed on the night of July 26th-27th, 1898, when on outpost duty, he received the brevet rank of Lieutenant Colonel, U. S. Volunteers, "for coolness under fire and efficient and meritorious services during the war."

Maj. Sackett remained with the Battalion until it was disbanded in New York City in September, 1898. He was not mustered out of service, however, until January, 1899, having been placed in command of a battalion of a Florida Regiment stationed at Huntsville, Ala.,

where he acted as Brigade Commander for a time, although only ranking as Major.

The following is an extract from the report of Col. Black, Chief Engineer, U. S. Volunteers, to the Adjutant General of the Army, dated August 22d, 1898, regarding the operations of the Battalion:

"For the drill, discipline and spirit, much credit is due the officers of the battalion, among whom discrimination cannot be made, excepting it be for the commanding officer, Major J. W. Sackett, 1st Fla. Vols., whom I have known for years as a valued civil engineer employee of the United States. It is but fair for me to say that, without his aid, I do not know how I could have performed the duties allotted to the Engineer force of the expedition."

After the war he rejoined the Florida Militia as Captain and was promoted through the various ranks until he was commissioned as Brigadier General of the Florida National Guard, on February 28th, 1906. Gen. Sackett was retired on February 22d, 1907, at his own request, due to press of work with the U. S. Engineer Department. During his service with the Florida Militia, he was a member of the Commission appointed to select a permanent camp site, personally located the site subsequently adopted by the State authorities, and until his death served on the Board which had charge of the grounds. He also supervised all the surveying and engineering work in connection with the development of the camp and target range, and prepared plans and estimates for its enlargement, water supply, etc. In 1917, when the United States entered the World War, Gen. Sackett immediately offered his services to the Adjutant General, U. S. Army, in any military capacity that would be useful.

On July 1st, 1918, in the line of duty, Gen. Sackett left St. Lucie Inlet, Fla., on board the dredge *Florida* bound for Jacksonville, where the boat was to be used in dredging operations at Camp Joseph E. Johnston then under construction. On the trip up the coast the dredge encountered a heavy gale. One of the bulkheads was stove in by a heavy wave on July 3d, 1918, and the *Florida* filled and sank within three minutes. Gen. Sackett, who was in the cabin on the upper deck at the time and, it is supposed, did not realize the danger, and two of the crew perished with the boat. He is survived by his widow and two sons, Arthur J. Sackett, Assoc. M. Am. Soc. C. E., and Guy W. Sackett.

Gen. Sackett was one of the best known and most popular men in Government work and military circles in Florida. He was held in high esteem by all who knew him, and his untimely death was a shock to his many associates and friends who were numbered by his acquaintances.

In the course of thirty years or more of service with the U. S. Engineer Department, he had been called on to investigate, discuss,

and make out in technical detail and report on hydrographical and topographical surveys, watersheds, run-off, stream flow, tidal currents, wave action, jetties, breakwaters, locks, etc. For the purpose of observation and study, he had visited the Panama Canal during its construction. He had also visited Havana, Cuba, for the purpose of observing the raising of the wreck of the U. S. S. *Maine,* as well as other works of improvement affecting waterways, drainage, and reclamation work throughout the United States.

As shown by his military record, Gen. Sackett had been identified with the Florida State Militia for many years, and was Past Department Commander of the United Spanish War Veterans, Department of Florida. He was also prominent in Masonic and Elk circles, and other fraternal organizations.

Gen. Sackett was elected a Member of the American Society of Civil Engineers on May 2d, 1894.

HERMANN FREDERICK AUGUST SCHUSSLER, M. Am. Soc. C. E.*

DIED APRIL 27TH, 1919.

Hermann Frederick August Schussler was born in Rastede, Germany, on August 2d, 1842. He came to California from Europe in 1864, after having spent two years in the study of hydraulic and mechanical engineering at the Zurich, Switzerland, and Karlsruhe, Germany, Polytechnic Schools. He gained his first practical experience on engineering work during vacations, having been employed on railroad and bridge work at Lucerne, Switzerland, and as a Mechanical Draftsman at Bremen, Germany.

As early as October, 1864, Mr. Schussler entered the service of the Spring Valley Water-Works (now the Spring Valley Water Company) which supplies water to San Francisco, Cal. He was advanced to be Chief Engineer in 1866, and served in that capacity for 42 years, until 1908, when he was relieved from active charge; he was retained by the Company in an advisory capacity, and continued in private practice as a Consulting Engineer.

During his long connection with the Spring Valley Water Company, Mr. Schussler was more than Chief Engineer of the Company's system. It fell to him to forecast the future water requirements of San Francisco and to take such measures from time to time as would put the Company in a position to meet the constantly growing demand for water. One near-by source after another was added to the System and, by the acquisition of large areas of land in the water-sheds, tributary to the storage reservoirs on which the supply to the city is dependent, the quality of the water was safeguarded. Ownership of the water-sheds has made it a simple matter to police the areas in which pollution would otherwise be possible.

During his 42 years of active charge as Chief Engineer of the Spring Valley Water Company, Mr. Schussler designed and constructed all the works of the Company, the aggregate cost of which during that time reached about $20 000 000, and which include such notable achievements as the Crystal Springs Dam, a gravity section concrete structure, and a number of lines of riveted wrought-iron pipe, the service of which for 30 to 40 years, with only moderate deterioration, is most gratifying evidence of the care with which the material for their construction was selected and with which their construction was supervised.

As early as 1870, in the capacity of Consulting Engineer to a mining company, Mr. Schussler designed a 30-in. pipe line, an inverted siphon, more than 3 miles in length, across Feather River Canyon, the success of which went far to give engineers confidence in the use of riveted

* Memoir prepared by C. E. Grunsky, G. A. M. Elliott and F. C. Herrmann, Members, Am. Soc. C. E., a Committee of the San Francisco Section of the American Society of Civil Engineers.

wrought-iron pipe. He was Engineer for the La Grange Mining Company in 1871-72, constructing for this company the mining canal from near Dom Pedro Bar, on the Tuolumne River, to near La Grange; he made surveys and triangulations for the control of the alignment of the Sutro Drainage Tunnel; he constructed the earth dam for the Lagunitas Storage Reservoir in Marin County, California, as well as the 7-mile, 12-in. pipe line for the water supply of Virginia City, Nev., in 1872-73. His activities extended to the Hawaiian Islands where he was Consulting Engineer for the Haiku Ditch Company, Island of Maui, and constructed about twenty wrought-iron, riveted, asphaltum-coated pipe lines on that island. He designed the sewer system for Coronado Beach, and was consulted on many other engineering problems.

In 1904, Mr. Schussler was asked by the late President Roosevelt as to his availability to become a member of the Isthmian Canal Commission and, a year later, in 1905, he was appointed, but did not serve, on the Board of Consulting Engineers which was named by the President to consider the question of whether the Panama Canal should be built as a sea-level canal or should be of the lock type.

The last notable structure which was carried to completion under Mr. Schussler's direction was the dam across the Klamath River, California, erected for the California-Oregon Power Company, and generally referred to as the "Copco" Dam. This is a concrete structure of the gravity type, notable for the fact that it closes a narrow subsurface gorge, 120 ft. deep, from which a gravel deposit had to be removed. The dam, in the main, was constructed on the interlocking block system, of quite dry concrete, requiring ramming — a system positive in results and much favored by Mr. Schussler.

On all the work in his charge, he was never satisfied with any that would just pass, that was "good enough". He insisted that it must be "first class", and the results he obtained speak for themselves.

To those who worked with him, and to all with whom he came in contact, he was kind and considerate, and he was successful in commanding a loyalty of subordinates that could only be the result of respect and appreciation of his honesty of purpose and his affectionate and sympathetic nature. He remained vigorous in body and mind to the day of his death.

Mr. Schussler was a good citizen, a thorough believer in American ideals, and enjoyed association with his fellow men. He was very fond of European travel, having made some twenty trips abroad, where he kept up his affiliations with the prominent engineers of England, France, and Italy.

He was married to Miss Louise A. Menges, on March 11th, 1869, and is survived by his widow and three children, Mrs. Martin Preuss, Alice C. Schussler and Dr. Hermann Schussler, Jr.

Mr. Schussler was elected a Member of the American Society of Civil Engineers on February 4th, 1914.

NAOJI SHIRAISHI, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1919.

Naoji Shiraishi was born at Tochi, a village in Kochi Prefecture, Japan, on October 29th, 1857. When he was in his teens, Japan was passing through the crisis preceding the Restoration of the Imperial Rule, and in those days there was more call for the arts of war than for those of peace. He early showed his far-sightedness in applying himself to the study of European languages, in utter disregard of the general cry for the need of warlike training for boys of his age. In 1870 he went to Yokohama and studied English under the Rev. S. R. Brown. In 1875 he entered the Kaisei Gakko, the predecessor of the present Tokyo Imperial University, for the study of Civil Engineering, and was graduated from that institution in July, 1881.

In 1883 he was sent by the Government to the United States to complete his study in that branch of knowledge. Entering Rensselaer Polytechnic Institute, he studied mostly under William H. Burr, M. Am. Soc. C. E. When Professor Burr took service with the Phœnix Bridge Company, Mr. Shiraishi accompanied him and was cordially given the freedom of both office and works to make a special study of bridge construction. He was deeply influenced by the personality and erudition of Professor Burr, and an intimate friendship grew up between the two men. Later, Mr. Shiraishi entered the service of the Pennsylvania Railway Company, to acquaint himself with the practice of railway engineering. Subsequently, he left America for Germany, and, while staying in Berlin, attended the lectures of Professor Winkler, in the Technische Hochschule in that city.

Returning to Japan in 1887, from a tour through the various countries of Europe, Mr. Shiraishi was appointed a Professor in the College of Engineering of Tokyo University. While holding that position, he acted as an adviser to the then organized Kansai Railway Company, in the location and construction of its lines. In those early days of railway construction in Japan, when the lines in operation in the country totaled not more than 1 000 miles, the services of expert engineers were not easily obtainable. The difficult nature of the work on the Kansai Railway having required the exercise of the highest technical knowledge, Mr. Shiraishi was called from his academic chair to give his undivided attention to the task, and, on his resignation of the professorship in 1890, he was made the first President and Chief Engineer of the Company. As Chief Engineer, he gathered around him, as his assistants, many of his former pupils at the University, and carried to a successful issue one of the most difficult railway works in the country. This achievement had far-reaching consequences, in that those young men whom he had trained in the classroom and in

* Memoir prepared by I. Hiroi, M. Am. Soc. C. E.

the field went forth as competent engineers and distinguished themselves in various works of importance with which they were entrusted later.

In 1891 he had conferred on him the degree of Kogaku Hakushi, the title of the highest academic honor in Japan for the profession. In 1898, he resigned his position with the Kansai Railway Company, the construction work of the latter having been completed.

In 1899 Mr. Shiraishi was appointed a member of the Railway Council, the highest advisory board for discussing matters relating to the railway systems in Japan. In the same year, at the request of the Kyushu Railway Company—another company in which he was interested—he again visited Europe and America to make investigations of the methods of shipping coal in those countries. He selected the Brown coal hoist, as the one best fulfilling the requirements of the Company, and, having made a purchase of three machines, each of 5 000 tons capacity, erected them at the terminal yard of the Company.

In 1910 he was made President of the Wakamatsu Harbor Improvement Company, and continued in that position for nine years, during which time he carried out the construction of breakwaters and quay walls, and extensive dredging of the harbor. Under his able guidance, the harbor has become the most important coaling port in Japan, the quantity of coal annually seeking outlet at the port reaching 7 000 000 tons. Along the shore of the harbor stand the Government Steel Works and other large factories, and the busy shipping traffic has transformed the waste flat of former years into an excellent harbor, crowded with sailing ships and steamers.

At about that time, Mr. Shiraishi acted also as Consulting Engineer to the Mitsubishi Company, and designed and constructed breakwaters, quay walls, metallic piers, warehouses of reinforced concrete, dry docks, etc., for that Company at Nagasaki and Kobe. The 7 000-ton floating dock built at the Kobe Shipyard and the use of caisson blocks of reinforced concrete for the breakwaters and quay walls there, were the first engineering feats of the kind in Japan.

In 1907, at the request of the Corean Government, he planned a system of railway lines in the southwestern part of the Peninsula. The present lines between Taiden, Gunzan, and Moppo are substantially the realization of his plans.

From 1911 to 1919 Mr. Shiraishi was Managing Director of the Inawashiro Hydro-Electric Power Company, which has for its work the utilization of the waters of Lake Inawashiro, 140 miles from Tokyo, for the development of electric power. The lake is 44 sq. miles in area and is 1 000 ft. above sea level. An average volume of water of 840 cu. ft. per sec. is now utilized, in two stages, producing electric power aggregating 92 000 h. p., which is transmitted to Tokyo at a pressure of 115 000 volts. The works are now being enlarged by utiliz-

ing the tail-water, so that before long the total output will amount to 130 000 h. p.

Mr. Shiraishi was married in 1882 to Miss Kikuko, a daughter of Mr. Tsuna Takenouchi, a prominent financier. He is survived by his widow and two sons. The elder of these sons follows his father's profession as a civil engineer, and the younger is an electrical engineer.

It is the lot of but few men to be able to accomplish so much in life as Mr. Shiraishi. His practice covered almost all branches of civil engineering, and the works in which he was engaged were of such magnitude as has scarcely been equalled by any in Japan.

Mr. Shiraishi will long be remembered, not only for his achievements as an engineer, but also for his personal qualities. His admirable character made him popular with all who came in contact with him. Although he was not a professed Christian, he was so much inclined to that belief that his interment was made according to its rites.

Mr. Shiraishi was a Member of the Institution of Civil Engineers of Great Britain. In January, 1919, he was elected President of the Japan Society of Civil Engineers. Beside being an engineer of the foremost rank, he was no less a politician, having occupied a seat in the House of Representatives of the Imperial Diet, to which he had been returned three times, and, as a member of the Seiyukai party, was a prominent figure in political circles.

Mr. Shiraishi was elected a Member of the American Society of Civil Engineers on October 4th, 1899.

HARRY GODFRED SIDENIUS, M. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1918.

Harry Godfred Sidenius was born in New York City on June 21st, 1881, and was educated at the Technical School of Copenhagen, Denmark.

Between 1904 and 1907 he was employed as Assistant and Chief Assistant to William Ingham, Chief Engineer on the design and construction of the water-works system at Port Elizabeth, Cape Colony, South Africa.

In 1907 Mr. Sidenius moved to Canada, and, during 1908-09, was engaged with Messrs. Smith, Kerry and Chace, Engineers on the City of Winnipeg power projects. From 1909 to 1913 he was employed in the Natural Resources Department of the Canadian Pacific Railway Company, chiefly as Resident Engineer of Construction of the Bassano Dam, on the Bow River, Alberta, one of the most noted structures of its type which has ever been built.

During 1914 and 1915 Mr. Sidenius was engaged as Assistant to the City Engineer of Calgary, Alberta, chiefly in connection with the studies and design of that city's water supply. During 1916 and part of 1917 he was attached to the Fourth Field Troop, Canadian Engineers, with rank of Captain, as Instructor at Sarcee Camp, Calgary, Alberta, Military District No. 13, in "Military Engineering and Trench Warfare." For 18 months previous to his death, he was Assistant to the Chief Engineer, Natural Resources Department, Canadian Pacific Railway Company, at Calgary, Alberta.

Mr. Sidenius was married in 1912 at Montreal, Que., Canada, and is survived by his widow and two children aged 4 and 6. His kindly and unassuming nature, his sterling integrity, and his ability as an engineer endeared him to all whose privilege it was to know him.

He was elected to membership in the Engineering Institute of Canada in May, 1915, and always took an active interest in the affairs of the Calgary Branch.

Mr. Sidenius was elected an Associate Member of the American Society of Civil Engineers on January 2d, 1912, and a Member on June 24th, 1914.

* Memoir prepared by A. S. Dawson, M. Am. Soc. C. E.

RAPHAEL CHART SMEAD, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1919.

Raphael Chart Smead was born at Washington, D. C., on October 25th, 1859. His father, the late Capt. J. R. Smead, was a graduate of West Point, a soldier in the Civil War, and was killed in the Battle of Bull Run. His grandfather, the late Capt. R. C. Smead, Corps of Engineers, U. S. Army, who was also an Army officer, was killed in the Mexican War.

After his preliminary education in the public schools, Mr. Smead studied under a private instructor for one year.

In 1878, he was employed as Rodman and Instrumentman making railway surveys on the Union Pacific Railway, and remained on this work until 1880. Between 1880 and 1881, he was engaged as Instrumentman on the Baltimore and Ohio Railroad; between 1881 and 1882, as Assistant and Resident Engineer on construction on the Norfolk and Western Railroad, between 1882 and 1883, as Resident Engineer on the Grafton and Greenbrier Railroad; and between 1883 and 1885, as Assistant and Resident Engineer on the Baltimore and Ohio Short Line Railroad.

In May, 1885, Mr. Smead was appointed United States Assistant Engineer in the office of the Washington Aqueduct, District of Columbia, where he remained until August, 1905. During that time, he was engaged on the work of building the dam at Great Falls and the tunnel between the Georgetown and McMillan Park Reservoirs, and as Superintendent of Maintenance and Operation of the Washington Aqueduct.

In August, 1905, he was transferred to the Dallas Engineer District where he was employed as U. S. Assistant Engineer until October, 1917. From October, 1917, to February, 1919, he served as U. S. District Engineer, at Galveston, Tex., and from that date until November 28th, 1919, as Principal Assistant Engineer of the Galveston District.

In 1918, Mr. Smead was commissioned a Major in the Engineer Officers' Reserve Corps, but was never called to active duty.

He was a man of pleasing personality and social disposition, who made friends easily and was liked and respected by all his associates. He died very suddenly while engaged on his regular official duties and is survived by his wife and one daughter.

Mr. Smead was elected a Member of the American Society of Civil Engineers, on February 1st, 1905.

* Memoir prepared by E. D. Hardy, M. Am. Soc. C. E.

EDWIN FOSTER SMITH, M. Am. Soc. C. E.*

DIED JANUARY 26TH, 1920.

Edwin Foster Smith was born in Catawissa, Pa., on August 18th, 1841. He was graduated from Union College, Schenectady, N. Y., in 1862, with the degree of A. B., and received the degree of C. E. from the same college in 1886.

His father, James Foster Smith, served the Schuylkill Navigation Company and its affiliate, the Philadelphia and Reading Railroad Company, in important positions, from 1843 until his retirement from engineering work in 1885.

Edwin Foster Smith spent his entire life of engineering activity—forty-nine years—in the service of the same two companies, entering that of the Railroad Company as Rodman and Leveler in 1862, and that of the Navigation Company as Assistant Engineer, under his father, in 1865, continuing in their service until his retirement, at the age of seventy, on September 30th, 1911. The activities of father and son, in the service of these two companies, thus covered a period of sixty-eight years.

On March 1st, 1876, Mr. Smith was made Chief Engineer of Canals for the Schuylkill Navigation Company and the Susquehanna and Tidewater Canal Company, and on February 25th, 1885, Superintendent and Engineer of Canals of the Philadelphia and Reading Railroad Company, Lessee.

From 1891 to 1895 he was engaged as Engineer in Charge of Construction of the Reading Terminal and the Terminal Railroad, at Philadelphia, Pa.; and from that time until his retirement in 1911 he acted as Superintendent of the Reading Terminal and Power Houses.

During much of his term of service with the two companies mentioned, Mr. Smith was also engaged in private engineering work, largely in connection with hydraulic operations. This work included the reconstruction, in 1876, of the timber dam, 6 843 ft. long, across the Susquehanna River, at Columbia, Pa.; consulting practice in connection with the water supplies of Reading and Norristown, Pa.; a report on the Castle Wood Dam near Denver, Colo.; the design of hydro-electric power plants near Delta, Pa.; the dam of the Concord Water Power Company on the Merrimac River at Concord, N. H.; and the dam of the Sissiboo Falls Paper Company at Weymouth, Nova Scotia. In 1891, Mr. Smith submitted, for the Philadelphia and Reading Railroad Company, a plan for the utilization of certain of the plants of that Company for the extension of the Philadelphia water supply.

* Memoir prepared by John C. Trautwine, Jr., Assoc. Am. Soc. C. E.

He served as Consulting Engineer and Referee in a number of cases involving testimony on engineering matters in the Pennsylvania and the United States Courts, including the condemnation proceedings of the U. S. Government in taking over the slack-water navigation of the Monongahela River, from Pittsburgh to Geneva, Pa., eighty-nine miles in length.

In 1901, Mr. Smith was engaged as Consulting Engineer to the Board of Engineers of the Aqueduct Commission of New York City, examining the construction of the new Croton Dam and Jerome Park Reservoir.

During the Civil War he served in the Pennsylvania State Militia and in the 26th Regiment, U. S. Emergency Volunteers, and, later, in the Government service of the Philadelphia and Reading Railroad.

He was a member of the Franklin Institute of Pennsylvania, a member and Past-President of The Engineers' Club of Philadelphia, and a member of the University Club of Philadelphia.

It was the writer's good fortune to enjoy friendly and professional intercourse with Mr. Smith, who was a kindly and always courteous gentleman, with a quiet and charming sense of humor.

Mr. Smith was elected a Member of the American Society of Civil Engineers on June 5th, 1895.

FRED CASWELL STANTON, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1919.

Fred C. Stanton, son of Fred O. and Rebecca Stanton, was born at Vineyard Haven, Mass., on June 7th, 1880. He attended the Vineyard Schools, and was graduated from Marion Academy and, later, from Dartmouth College, taking both the B. A. and C. E. degrees.

During one of his vacations, while attending the Thayer School, Mr. Stanton was employed at Cape May, N. J., on the harbor protection work, under the Engineer Corps, United States Army.

After his graduation, he was employed by the same Department on the improvements of the Brazos River, Texas, under Maj. Jadwin, U. S. A. He was stationed at Houston, Galveston, and at various points along the Brazos River. He was also employed on the Rio Grande River, on the Texas border, and built the ship channel between Houston and Galveston.

When Gen. George W. Goethals, M. Am. Soc. C. E., was put in charge of the construction of the Panama Canal, Mr. Stanton as Civilian Engineer was engaged on concrete construction and dredging in the Canal Zone. Later, he was advanced to the position of First Assistant Engineer of the Atlantic Division of the Panama Canal, under his former chief, Maj. Jadwin, and, later, under Gen. W. L. Sibert, U. S. A., M. Am. Soc. C. E. He designed and constructed the harbor works for the protection of the new harbor at Cristobal, including both the great breakwater at Toro Point and the Eastern Breakwater.

After some eight years of service in the Canal Zone, his health, and also that of Mrs. Stanton, was affected by the tropical climate, and this, with the desire to engage in business for himself, led him to resign from the Canal service in March, 1914.

Mr. Stanton returned to the United States and organized the Stanton-White Dredging Company, which company purchased a number of dredges and engaged in work on reclamation projects in the Mississippi Valley. This work continued until 1918, when labor troubles, occasioned by the draft, considerably restricted the operations of the firm.

On the entrance of this country into the World War, Mr. Stanton was very desirous of enlisting in one of the active branches of the Army, but on the advice of friends who knew his qualifications as an engineer and his familiarity with maritime affairs, he was offered, and accepted, a position with the United States Shipping Board, Emergency Fleet Corporation, as District Plant Engineer of the Southern District, with

* Memoir prepared by J. M. Clack, Assoc M. Am. Soc. C. E., and R. B. Kyle, M. Am. Soc. C. E.

headquarters at Jacksonville, Fla. He was enthusiastically engaged in this work, when stricken with influenza in January, 1919, which developed into pneumonia, and he passed away at the hospital, in Jacksonville, on January 24th, 1919.

Mr. Stanton was married to Elizabeth Stokes, of Jersey Shore, Pa., on October 25th, 1909, who survives him. Mrs. Stanton always accompanied him to all the positions he held from the date of their marriage.

His organization in the Emergency Fleet Corporation and his many friends are left to mourn the loss of a real executive and a true friend.

Mr. Stanton was elected an Associate Member of the American Society of Civil Engineers on April 1st, 1908, and a Member on December 31st, 1913.

HENRY MORTON STONE, M. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1918.

Henry Morton Stone, the eldest son of the late John William and Hannah Ann (Cowpland) Stone, of Glen Loch, Pa., was born in West Nantmeal Township, Chester County, Pa., on April 28th, 1858.

He was educated at Treemount Seminary, Norristown, Pa., where he studied engineering under the late Dr. John W. Loch. His earliest engineering experience was with railroads in the East, but the greater portion of his life was spent in the service of Western roads, or in the employ of Chicago interests.

Mr. Stone's first work was with the Pennsylvania Railroad, Eastern Lines, where he was engaged in the location and construction of the road from Whitings to Bay Head Junction, N. J., and of the "Cut Off" at Lancaster, Pa.

He resigned from the Pennsylvania Railroad to accept a position with the Denver and Rio Grande Railroad, having had charge of the construction of a section of that road in the Rocky Mountains near Dillon, Colo. With the temporary abandonment of this work, he accepted a position with the St. Louis, Iron Mountain and Southern Railroad, and, later, returning to his native State, was employed by a corporation known as the Delaware River, Phoenixville, and Lancaster Railroad to locate a railroad through the northern part of Lancaster and Chester Counties. He resigned to accept a position with the Lehigh Valley Railroad Company at Sayre, Pa. While with the Lehigh Valley, he was engaged in making surveys for the entrance of that railroad into Buffalo.

In 1883, Mr. Stone again went West, and accepted a position with the Chicago and Northwestern Railroad, with which he remained for more than five years, his principal work being the construction of new lines in the Northern Peninsula of Michigan.

After leaving the Northwestern, he was for a time Engineer of the Cairo Land and Construction Company, and in that capacity had charge of the construction and maintenance of the dikes at Cairo, Ill. Mr. Stone resigned this position to enter the service of the Chicago, Milwaukee and St. Paul Railroad, where he had charge of double-tracking a considerable section of that road near Marion, Iowa. His next position was with the municipality of Wilmette, Ill.

In 1896, Mr. Stone was sent by a Chicago syndicate of capitalists to Cape Breton, Nova Scotia, to locate and build a railroad (with docks and terminal facilities) from the Western Coast to the coal fields. In 1900, he became Location Engineer for the Algoma Central Railway,

* Memoir prepared by J. W. Stone, Esq.

in which capacity he located a considerable portion of the present line of that road. Later, he was appointed Bridge Engineer of the same corporation.

On January 1st, 1903, he entered the service of the Chicago, Rock Island and Pacific Railroad, with which corporation he remained until his death, at different times holding the positions of Assistant Engineer at Lincoln, Nebr., Division Engineer at Fairbury, Nebr., and Little Rock, Ark., Location Engineer in charge of location and construction in Texas and Louisiana, District Engineer of the Northern District at Cedar Rapids, Iowa, and of the Southwestern District at Topeka, Kans., and Chief Engineer of the Chicago, Rock Island and Gulf at Fort Worth, Tex. With the abolishment of that position he was for a time with the Valuation Department, and, at the time of his death, was Engineer of Construction in charge of extensive double-track work authorized by the Federal Administration, at Trenton, Mo.

Prior to going to Trenton, Mo., Mr. Stone had been loaned by the Rock Island Railroad Company to the Union Pacific Railroad to take charge of the field work of the reconstruction of the Grand Island Bridge crossing the Missouri River at St. Joseph, Mo.

He was married in Chicago, on September 11th, 1889, to Miss Henrietta Reigart, daughter of Adam W. and Mary Bender Reigart, formerly of Lancaster County, Pa.

He is survived by his widow, residing at Kansas City, Mo., and one son, Morton Reigart Stone, Superintendent of the Idaho Fruit Products Company at Meridian, Idaho.

Mr. Stone was elected a Member of the American Society of Civil Engineers on April 3d, 1907.

ARNOLD HENRY SUTERMEISTER, M. Am. Soc. C. E.*

DIED JUNE 30TH, 1918.

Arnold Henry Sutermeister was born at Fort Wayne, Ind., on November 16th, 1869. His father, the late Arnold Sutermeister, who was born in Switzerland, was a prominent contractor in the West, having been engaged in business in Fort Wayne, Ind., and Kansas City, Mo. He was a Captain in the Civil War, and commanded the Eleventh Indiana Battery, which command he himself raised at Fort Wayne. This battery shared, to a great extent, in the hottest fighting of the War. Capt. Sutermeister was also with Sherman in his famous march to the sea. Mr. Sutermeister was always very proud of his father's military record.

Mr. Sutermeister spent his early boyhood and received his schooling at Fort Wayne, and, during his early 'teens, went to Kansas City, where he attended High School. J. A. L. Waddell, M. Am. Soc. C. E., a personal friend of his father, was instrumental in sending the young man to Rensselaer Polytechnic Institute. He took the examinations for the Institute in Mr. Waddell's Kansas City office.

He took the civil engineering course at Rensselaer, was graduated in June, 1892, and then entered the service of the United States Geodetic Survey, being engaged as Transitman on the survey of the Hudson River until the fall of that year, when he accepted a position as Assistant Division Engineer on the Mohawk Division of the New York Central and Hudson River Railroad, at Albany, N. Y. He was quickly promoted to Division Engineer of the same Division, and retained that position until July, 1894, when he entered the employ of the Troy Union Railroad, at Troy, N. Y., first as Roadmaster and then as Superintendent and Engineer, in which capacity he had charge of the design and erection of railroad masonry and bridges.

In July, 1898, Mr. Sutermeister was appointed Engineer in Charge of the Elimination of Grade Crossings for the Board of Railroad Commissioners of New York State, later known as the Public Service Commission, Second District. This position he held until his death, which occurred on June 30th, 1918, at Albany, as the result of an unusual accident in which he sustained a fracture of the skull, due to a blow received when a demountable rim flew from the wheel of his automobile while he was adjusting it. His long service of 20 years as Engineer of Grade Crossings of the Public Service Commission, speaks for the quality of his professional work.

Some of the structures of the grade crossing work, with the design and execution of which he was directly connected, were: The Chenango

*Memoir prepared by J. W. Pfau, M. Am. Soc. C. E.

Street Viaduct, at Binghamton, over the Erie and Delaware, Lackawanna and Western Railroads; the elevation of the New York Central tracks through Schenectady and through Yonkers; the elimination of the grade crossing at North Pearl and Von Woert Streets on the main line of the New York Central Railroad, in Albany, and also many other grade crossing eliminations throughout the State, from three hundred to four hundred works of this kind having been built during the period in which Mr. Sutermeister served the Commission.

In 1895 Mr. Sutermeister was married to Miss Caroline M. Kreischer, of Albany, who, together with two sons, Arnold, age 19, and Karl, aged 16, survives him. He was a man of pleasing personality and sterling character, and was possessed of particularly sound judgment. His most pronounced characteristic was his sense of fairness. He not only had the desire but also the ability and determination to be fair, a particularly desirable trait for one in the position he held, where engineering decisions had to be made to meet, as nearly as possible, the views and desires of all the various interested parties, namely, adjoining property owners, municipalities, and railroad companies. Mr. Sutermeister was of a modest and retiring disposition, and was a firm believer in the simple life.

He was a member of the University Club of Albany, Sons of Veterans, and the Rensselaer Society of Engineers, and had served as President of the Eastern Division of the Rensselaer Alumni Association.

Mr. Sutermeister was elected a Member of the American Society of Civil Engineers on June 3d, 1902.

CARL UHLIG, M. Am. Soc. C. E.*

DIED MAY 4TH, 1919.

Carl Uhlig was born in Chemnitz, Saxony, on December 31st, 1846. He was educated in Saxon schools and was graduated from the Polytechnic School in Dresden in 1867.

He came to the United States in 1868 and shortly afterward entered the service of the United States Coast and Geodetic Survey as Aid, having been engaged in work at various points on the coast of California, principally at San Pedro and Crescent City.

In 1876, Mr. Uhlig entered the employ of the Board of State Harbor Commissioners in San Francisco as Chief Draftsman, being later appointed Assistant Engineer, and remained in that position continuously for 43 years, until his death. During that time he was connected with the design of the numerous bulkhead wharves, piers, ferry slips, sea walls and other water-front structures built by the Board, and accumulated a fund of information bearing on the hydrographic and topographic conditions of the San Francisco frontage that was particularly valuable and made of him an authority on this subject, consulted by all who contemplated any engineering work in that vicinity.

Mr. Uhlig was a man of high character, scrupulous integrity, and amiable personal qualities. He had a wide circle of friends in all walks of life and was regarded with feelings of esteem and affection by all who enjoyed the privilege of his friendship. The best tribute to his ability as an engineer is the fact of his having been retained in his position by his employers, the Board of State Harbor Commissioners, during the political vicissitudes of over 40 years; though the personnel of the Board was changed repeatedly, his thorough knowledge of conditions was considered invaluable and no attempt was made to replace him.

Mr. Uhlig never married, and is survived by two sisters. He had been in failing health for some years, and his death from heart disease on May 4th, 1919, was not entirely unexpected.

Mr. Uhlig was elected a Member of the American Society of Civil Engineers on October 7th, 1903.

* Memoir prepared by the following Committee of the San Francisco Section of the American Society of Civil Engineers: Frank G. White, Howard C. Holmes, and Jerome Newman, Members, Am. Soc. C. E.

GEORGE WESTON, M. Am. Soc. C. E.*

DIED JANUARY 7TH, 1920.

George Weston was born in Kalamazoo, Mich., on January 30th, 1861.

He began his professional career in 1880 when he entered the Engineering Department of the Missouri, Kansas and Texas Railway as Rodman, serving in this position until 1883.

From 1885 to 1887, he was employed as Rodman, Instrumentman, and Assistant Engineer in charge of construction with the Gulf, Colorado and Santa Fé Railroad.

In 1887, Mr. Weston established relations with the late Charles T. Yerkes, of Chicago, Ill., and was engaged until 1896, as Engineer in charge of the construction of a number of miles of cable and electric street railway track in that city.

From 1896 to 1901, he served as Manager of the Construction Department of Naugle, Holcomb and Company of Chicago, Ill., having been engaged as follows: In 1896 and 1897, as Chief Engineer on the construction of the "Suburban" Railroad, Chicago; in 1898, he made examinations and reports on steam railroad and electric properties for the firm; and from 1898 to 1901 he had full charge as General Manager and Chief Engineer of the construction of 66 miles of the Tennessee Central Railroad.

From January, 1901, to 1907, Mr. Weston was associated with his brother, Charles V. Weston, M. Am. Soc. C. E., and also with Bion J. Arnold, M. Am. Soc. C. E., and the Arnold Company, of Chicago, in practice as a Consulting Engineer on valuation work and in making reports pertinent to electric traction operations.

When the Board of Supervising Engineers of the Chicago Traction Company was organized in 1907, Mr. Weston was appointed Assistant Chief Engineer and, at a later date, became the representative of the city on that Board and Engineer for the Board which latter position he retained until January, 1919. At the time of his death he was President of Weston and Company, Consulting Engineers, of Philadelphia, Pa., having been retained as Consulting Engineer for the Philadelphia Rapid Transit Company. He was also conducting work in connection with valuation matters for the Public Service Railway of New Jersey.

Mr. Weston died suddenly in Philadelphia, Pa., on January 7th, 1920, from acute indigestion, and is survived by his widow and two brothers, Charles V. Weston, M. Am. Soc. C. E., and Joseph Weston.

He was one of the best known experts on electric street railway affairs in the United States, and his work in connection with the construction and electrification of railroads and street railroads was exten-

* Memoir prepared by Horace L. Howell, Esq., New York City.

sive throughout the Middle West. He had also been active in questions of valuation and industrial relations and had written many papers in reference to these subjects.

Mr. Weston was admired by his associates for his qualities of capability, loyalty, fairness, and the intense energy which characterized his work.

He was a member of the Western Society of Engineers, the American Electric Railway Association, for which, in connection with his work on its Committee on Valuation, he had just completed a report outlining procedure and methods of deriving and attaining fair value for electric railway properties, the Engineers' Clubs of New York City and Chicago, and the Chicago Athletic Club. He was also active in Masonic circles.

Mr. Weston was elected a Member of the American Society of Civil Engineers on June 5th, 1907.

RUFUS MASON WHITTET, M. Am. Soc. C. E.*

DIED DECEMBER 10TH, 1918.

Rufus Mason Whittet, son of Alexander and Isabella (Proudfoot) Whittet, was born on October 17th, 1878, in Lowell, Mass. He came of Scotch ancestry, being a lineal descendant of James Whittet, of Kintillo, Scotland.

He was educated in the Lowell public schools, having been graduated from the Moody Grammar School in 1893 and from the Lowell High School in 1897, at the age of nineteen. Deeming it wise to continue his academic studies further, he returned the following year to the Lowell High School, taking the Post Graduate Course.

Mr. Whittet then passed the entrance examinations for both Harvard College and the Massachusetts Institute of Technology. Deciding in favor of Technology, he entered that institution in 1898 and was graduated in June, 1902, with the degree of Bachelor of Science in the Department of Sanitary Engineering, presenting as his thesis: "A Study of the Stream Flow Records of the United States Geological Survey."

After being graduated from the Massachusetts Institute of Technology, Mr. Whittet joined the Engineering Staff of the State Board of Health, and in the wide field of work covered by that Department he found employment congenial to his tastes. Taking a deep interest in its work, he soon became a most valued member of its staff, and was the Principal Assistant Engineer of the Department at the time of his death.

He was a civil engineer who believed in the elevation and dignity of his profession, and he acted on this belief with a fine enthusiasm. His principal characteristics, clear thinking and sound judgment, were combined with great executive ability, and his early death ended a career begun with great promise.

Mr. Whittet was a member of the Boston Society of Civil Engineers, the New England Water Works Association, and prominent in the Masonic Fraternity.

He was married in September, 1916, to Effie Osgood Byron, of Jamaica Plain, Mass., who, with a daughter, Helen Isabella, born on January 16th, 1918, survives him.

In appreciation of his service to the State, the State Department of Health adopted the following tribute to his memory:

"Rufus Mason Whittet, C. E., Principal Assistant Engineer of this Department, died on December 10th, 1918, a victim of the prevailing influenza epidemic.

* Memoir prepared by X. H. Goodnough, M. Am. Soc. C. E.

Mr. Whittet joined the Engineering Staff of the State Board of Health upon his graduation from the Massachusetts Institute of Technology in 1902, and has been the Principal Assistant Engineer of the Department since 1908.

His energy, thoroughness, clear thinking and sound judgment were of invaluable assistance in the studies of water supply and drainage and the great variety of kindred problems embraced in the work of the Department, while his quiet manner, his fairness, and his kindness made work with him a pleasure. These qualities, with a high sense of honor and loyalty to his associates, made him respected by all and loved by a wide circle of friends."

Mr. Whittet was elected an Associate Member of the American Society of Civil Engineers on May 2d, 1911, and a Member on November 4th, 1914.

ELLIOTT HINCKLEY WILSON, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1918.

Elliott Hinckley Wilson was born in New Orleans, La., on March 25th, 1851. He entered Washington University, where he made a remarkable record, ranking high in every class, and was graduated in 1871, as a Civil Engineer.

In 1870, Mr. Wilson served as Rodman on the St. Louis and South Eastern Railway, and during the next two years as Rodman, Topographer, and Leveler on the Northern Pacific Railway in Montana. In 1873, he was Assistant Engineer on the Cairo and Vincennes Railway, and, in 1874 and 1875, served in the same capacity on the location, construction, and maintenance of the St. Louis, Kansas City and Northern Railway.

During the next two years Mr. Wilson was Assistant Engineer on the Ohio and Mississippi Railway, and entered the service of the Mississippi River Commission in 1878 as Assistant Engineer at St. Joseph, Mo. He remained with this Commission until 1882, having charge of boring operations, under Maj. Suter, a work undertaken to determine the extent and character of the alluvial deposits of the Mississippi and Missouri Rivers. He also had charge of certain river improvements at Atchison, under Maj. Suter.

In 1882, Mr. Wilson accepted the position of Assistant Engineer on the Missouri Pacific Railway, taking charge of the location of the line between Atchison, Kans., and Omaha, Nebr. After that, he was in charge of location and construction for the Union Pacific Railway in Montana, Nebraska, and Idaho, until September, 1884.

At the latter date he formed a partnership with Mr. John Gillie, under the firm name of Wilson and Gillie and, until 1898, conducted a general civil and mining engineering practice at Butte, Mont. In 1898, Mr. Gillie entered the employ of the Anaconda Company, and a little later Mr. Wilson became affiliated with the Heinze interests, remaining with that organization until 1906. For three months in 1889 he acted as General Manager of the Bi-Metallic Mining Company.

Mr. Wilson was a United States Deputy Mineral Surveyor for Montana and Idaho, and for the last few years had devoted himself to private work as an examiner of mining properties for various interests.

He had served a term in the Montana Legislature, was elected a Member of the American Institute of Mining Engineers in 1887, and was one of the Charter Members of the Montana Society of Engineers, having been elected to the Presidency of that Society in 1891. He was

* Memoir prepared from information furnished by Eugene Carroll, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

one of the pioneer members of the Silver Bow Club of Butte, Mont., and made his home there.

Mr. Wilson suffered from a malignant affliction of the throat, and was with difficulty persuaded by his friends to seek relief in the hospital at Warm Springs. He died there on November 27th, 1918. He is survived by three sisters, Mrs. M. S. Smith, of Fort Collins, Colo., Mrs. W. Bradford, of Waverly, Ill., and Miss Josie Wilson, of Granville, Ill.

Mr. Wilson was elected a Member of the American Society of Civil Engineers on February 1st, 1888.

IRVING SPARROW WOOD, M. Am. Soc. C. E.*

DIED OCTOBER 20TH, 1919.

Irving Sparrow Wood, born in Providence, R. I., on November 27th, 1857, was the son of Charles P. and Sarah S. (Robinson) Wood. He was of New England parentage. His education was obtained from the public schools of his native city, supplemented by a course in Mowry and Goff's English and Classical School, a local institution, from which he was graduated in June, 1877.

Mr. Wood promptly took up the line of work which he was destined to follow during his life, entering the office of the City Engineer of Providence and working there in various capacities until the spring of 1881, when he was assigned to the Water Department, then in charge of the late Edmund B. Weston, M. Am. Soc. C. E. He soon became First Assistant, acting as such until May 1st, 1899, when he was appointed to the vacancy caused by the resignation of Mr. Weston. This position he held until his death on October 20th, 1919, which occurred in the house in which he was born.

During the twenty years of his administration, the population supplied by the Providence Water-Works increased from about 180 000 to nearly 300 000, and the annual consumption of water rose from about 3 500 000 000 gal. to nearly 8 000 000 000 gal. This expansion required a corresponding increase of the distributing pipes year by year and the installation of more ample and efficient pumping machinery. An extensive and highly successful filtration system was also introduced.

Mr. Wood was affiliated with the Boston Society of Civil Engineers, the New England Water Works Association, and the Providence Engineering Society.

He was married on October 14th, 1903, to Miss Margaret E. Read, who survives him.

Modest and unassuming in demeanor, Mr. Wood was diligent and conscientious in the performance of his official duties, courteous and fair with his subordinates, agreeable as an associate, and faithful and considerate as a friend.

Mr. Wood was elected a Junior of the American Society of Engineers on March 5th, 1890, an Associate Member on March 7th, 1900, and a Member on March 6th, 1906.

* Memoir prepared by Herbert E. Sherman, M. Am. Soc. C. E.

WILLIS BENTON WRIGHT, M. Am. Soc. C. E.*

DIED NOVEMBER 16TH, 1919.

Willis Benton Wright was born in Pittsfield, Mass., on February 25th, 1860. He was a son of Samuel Augustus Wright and Ann Maria (Butler) Wright. His father was the son of Samuel C. Wright and Olive (Benton) Wright, and traced his descent to James Wright of Milford and Durham, Mass. (1698). His mother, Ann Maria (Butler) Wright, born in Connecticut, was the daughter of Sylvester and Anna Butler, whose family traced its descent to Richard Butler, a deacon in the church of the Rev. Thomas Hooker. This is the church the members of which came from Boston and settled Hartford, Conn., in 1636.

Mr. Wright was prepared for college at the Middletown, Conn., High School and took the Civil Engineering Course at Sheffield Scientific School, Yale University, where he was a member of Phi Gamma Delta and received a Senior appointment.

After his graduation from Sheffield, Mr. Wright entered the service of Commander Garringe, U. S. N., the Consulting Engineer of Obelisk fame. In 1886, he was engaged on construction work with the Minneapolis, St. Paul and Sault Sainte Marie Railroad, and, later, was employed on the same kind of work in the Cascade Mountains for the Oregon Pacific Railroad, and, in Lower California, Mexico, for the Ferrocarril San Quintin y Yuma.

After two years more in the service of the Minneapolis, St. Paul and Sault Sainte Marie Railroad, and a year's travel, he was appointed, in 1895, Division Engineer of the Drainage Department of the Sewerage and Water Board of the City of New Orleans, La., which position he held at the time of his death on November 16th, 1919.

He was married on May 18th, 1896, in New Orleans, La., to Miss Juliette Pulver, daughter of David Barker Pulver, of that city, by whom he is survived. They had no children.

Mr. Wright had many business and professional qualities. Those who knew him always retained a high regard for his clear-sighted analysis of conditions, his high standard of fair dealing, his integrity, and his tactful treatment of all questions submitted to him. He was an engineer of unquestioned ability, active, energetic, and had endeared himself to all who knew him.

Mr. Wright was elected a Member of the American Society of Civil Engineers on November 3d, 1897. He was also a Charter Member and a Past-President of the Louisiana Engineering Society.

* Memoir prepared by Alfred F. Theard, M. Am. Soc. C. E.

EDGAR HAROLD ANNEAR, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 28TH, 1918.

Edgar Harold Annear was born at Ceres, Cal., on December 30th, 1881. He received his early education in the schools of Ceres and Modesto, and pursued a course in civil engineering at the University of California for two years. He then received an appointment to the West Point Military Academy, but was compelled to give it up at the end of a year because of the prolonged illness of his father.

On returning to California, Mr. Annear was appointed a Deputy United States Land Surveyor, made a survey of Government lands in Monterey County, and, later, engaged in the practice of surveying and irrigation engineering in Stanislaus County, California.

In 1906, he was elected County Surveyor of Stanislaus County, and was re-elected without opposition in 1910 and again in 1914.

In January, 1918, Mr. Annear resigned his office to take a commission in the Engineer Reserve Corps, U. S. A., and, after three months' training at American University, Washington, D. C., was sent to France in command of Co. B, 43d Regiment of Engineers, U. S. A. He had been promoted to the rank of Major, and had returned to the United States to take back combatant troops, when he was taken suddenly ill, soon after landing at Hoboken, N. J., and died there on August 28th, 1918.

During his term of office as County Surveyor, Maj. Annear was in charge of the planning and construction of a great number of roads and bridges, which were required by the rapid development of the county. He realized the ultimate desirability of permanent and esthetic structures, and the bridges of Stanislaus County are lasting monuments to his genius and constructive ability.

In the latter part of 1916, the people of Stanislaus County voted bonds for the construction of 126 miles of concrete highways. The successful promotion of this project was due, in a large degree, to the untiring effort with which Maj. Annear devoted himself to the education of the people to the necessity and ultimate economy of permanent roads, and, after the bonds were voted, he was placed in direct charge of their planning and construction. With an energy which was characteristic of him, he perfected a highway organization, completed surveys and plans, and succeeded in getting construction under way with such speed that, notwithstanding the ever increasing shortage of labor and material, he was able to see the completion of all but about 18 miles of the system before his resignation in January, 1918.

* Memoir prepared by D. M. McPhetres, Esq.

The highways which were built under his direction are conceded to be among the best in the United States.

Maj. Annear was an engineer of the highest integrity and ability. He kept constantly in touch with his work, was possessed of a strong grasp of detail, and the high class of workmanship which is evident in all his work is due largely to his own personal supervision. Under the most trying circumstances, he was always cheerful, was ever ready to overlook the errors of others, and his kindly smile and cheerful greeting made him beloved by his associates and all who knew him.

In July, 1909, he was married to Miss Margaret L. McFarland, who, with a daughter, survives him.

Maj. Annear was elected an Associate Member of the American Society of Civil Engineers on March 5th, 1912.

BERNT BERGER, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 16TH, 1919.

Bernt Berger was born in Drammen, Norway, on July 12th, 1866. He was graduated from the High School there in 1881, and from the Mechanical Engineering Branch of the Technical Institute of Trondhjem, Norway, in 1885.

After one year's employment in the City Engineer's Office in Drammen, Mr. Berger came to New York City, arriving on September 14th, 1886, and almost immediately entered the employment of the late Theodore Cooper, M. Am. Soc. C. E., Consulting Bridge Engineer, with whom he remained until the latter retired from business in 1908, when Mr. Berger succeeded to his very important consulting practice, in addition to what he had individually developed along similar lines.

Mr. Berger's great accuracy and careful methods of investigation and sound treatment of technical subjects won for him the complete confidence of Mr. Cooper and his clients, and he developed a great capacity for handling successfully difficult structural problems and had responsible charge of many reports, investigations, and designs. He specialized largely in important and long-span steel bridges, general steel construction, masonry and concrete arch spans, reinforced concrete construction, and substructure work.

Among the important structures with which Mr. Berger was connected may be mentioned the Quebec Bridge, the New York Public Library Building, the New York City Elevated Railroad structures, and important bridges for many railroad companies, including some in South America and in Japan. He also specialized in examinations and recommendations for the safety of existing structures and for difficult repair and reconstruction work that was of great value to the engineers, contractors, and architects who were his clients. He was associated with much of the recent New York City subway construction, having been retained to supervise the steelwork on several important sections.

Few of his clients failed to re-engage him whenever they had subsequent work in his field. He was employed frequently by the New York Edison Company and the Edison Electric Illuminating Company, of Brooklyn, The Dayton Light and Power Company, the Utica Gas and Electric Company, the Underpinning and Foundation Company, The U. S. Realty and Improvement Company, The Cranford Company, of Brooklyn, Holbrook, Cabot and Rollins, The Pennsylvania Shipbuilding Company, The New York Tunnel Company, and many other companies and individuals were included among his clients.

* Memoir prepared by Frank W. Skinner, M. Am. Soc. C. E.

Mr. Berger was a Past-President of the Brooklyn Engineers' Club, a member of the Engineers' Club of New York, the American Society for Testing Materials, and of the American Concrete Institute. He was also the Permanent Chairman of the Trondhjem Technical Club and a member of the Crescent Athletic Club, of Brooklyn, and, for many years, one of the Board of Managers and Treasurer of the Norwegian Lutheran Deaconesses Home and Hospital, a Trustee of the Norwegian Sailors' Church, Director of the Norwegian Society, and Secretary of the Hospital Saturday and Sunday Association, all of Brooklyn, N. Y.

Mr. Berger was fond of outdoor life and sports, including golf, sailing, bicycling, and skeeing. Although of a studious disposition, with cultivated literary taste, he found time for much social relaxation with a large circle of warm friends who claimed all his leisure moments. He was especially devoted to the interests of all his fellow engineers and never failed to lend a kindly hand and to have an encouraging word for the younger members of the Profession, many of whom he established in positions among his numerous friends and clients. His purse was always open to the necessities of struggling young engineers or students, and he befriended many of them, especially those who had arrived from abroad with only a few friends and no experience in this country. Mr. Berger lived in Brooklyn for twenty-five years; he was never married, and is survived by one brother in New York City and two brothers in Norway.

By his death on January 16th, 1919, the Society has been deprived of an able, vigorous member, always ready to devote his utmost efforts and zeal for its benefit; the Profession of Civil Engineering has lost a scholar, designer, and constructor; his clients have been deprived of notable skill and practical ability and unusually reliable services that were always rendered with the utmost care and thoroughness on every occasion; while his numerous friends can never forget the genial comradeship and hearty affection that endeared him beyond the ordinary to all with whom he came in close and sympathetic touch.

Mr. Berger was elected an Associate Member of the American Society of Civil Engineers on April 5th, 1893.

WILLIAM BLAIR BOVYER, Assoc. M. Am. Soc. C. E.*

DIED JULY 20TH, 1918.

William Blair Bovyer, the son of Edward Bovyer and Anna (Kearse) Bovyer, was born in San Francisco, Cal., on March 16th, 1887. He received his elementary education in the public schools of San Francisco and attended the Mission High School of that city, from which he was graduated in 1905. He then entered the University of California with the intention of studying law, but soon found, however, that engineering had a stronger appeal for him. He gave up the study of law and began a course in Civil Engineering from which he was graduated in 1910 with the degree of B. S.

For the first two years following his graduation, Mr. Bovyer worked, successively, for the Pacific Rolling Mill Company, Dyer Brothers Iron Works, the United Railroads of San Francisco, Selby Smelting Company, and Maurice C. Couchet, M. Am. Soc. C. E., Consulting Engineer.

In 1912, he became connected with the City Engineer's Office of San Francisco, where he attained the grade of Assistant Engineer. He worked on most of the city's important engineering works of that period and made many of the drawings and designs of the tunnels, sewer systems, water-works, and other structures constructed by the city.

In 1917, Mr. Bovyer was granted a leave of absence from the City Engineer's office and became connected with the work of the Board of State Harbor Commissioners of San Francisco, where he was employed until his death. During this time he was engaged in the design of all classes of water-front structures such as piers, wharves, buildings, etc.

Mr. Bovyer was a man who always put his best efforts into his work, and the record which he made proves that he made no mistake in his choice of a profession. He was a man of pleasing personality and in all his personal and professional relations his outstanding characteristics were kindness and complete sincerity and frankness. His joyous outlook on life and its many problems always acted as an inspiration and a tonic to all who came in contact with him. His genial nature will be long remembered by his friends.

In August, 1915, Mr. Bovyer was married to Miss Cora Niedor, who, with two daughters, Ruth, age four, and Patricia, age one, survives him.

Mr. Bovyer was elected a Junior of the American Society of Civil Engineers on October 1st, 1913, and an Associate Member on September 12th, 1916.

* Memoir prepared by the following Committee of the San Francisco Section of the American Society of Civil Engineers: Frank G. White, M. Am. Soc. C. E., and Leon H. Nishkian, Assoc. M. Am. Soc. C. E.

CHARLES EDWIN BRIGHT, Assoc. M. Am. Soc. C. E.*

DIED APRIL 3D, 1919.

Charles Edwin Bright was born in Sumner County, Tennessee, on December 27th, 1866. He was the son of Dr. Charles T. and Mary Branham Bright.

After attendance at the public schools of Sumner County, Mr. Bright entered Vanderbilt University at Nashville, Tenn., to take up civil engineering, but, in 1887, he had to give up college, after having completed three years of the four-year course.

From June, 1887, to February, 1888, Mr. Bright was engaged as Rodman in the City Engineer Department of Nashville, Tenn., and from February to November, 1888, as Transitman, Leveler, and Resident Engineer on the location and construction of dummy railways and general city surveying.

From January to May, 1889, he was employed on the Florence Northern Railway as Transitman, Leveler, and Assistant Engineer on location and construction in Alabama and Tennessee, and after making a survey for a highway bridge across Duck River, at Columbia, Tenn., in May, 1889, he served as Leveler (June, 1889) on the Louisville and Nashville Railway on the location of 10 miles of double track. In July and August of that year, Mr. Bright was again employed with the City Engineer Department of Nashville as Inspector of granite block pavement for the city. In September, 1889, he entered the employ of the East Tennessee Land Company and had charge of a party making a topographical survey for, and staking out, the Town of Harriman, Tenn., as well as the preliminary surveys and location for the Harriman and Brushy Mountain Railway, on which he also served as Resident Engineer on construction.

From December, 1891, to October, 1895, Mr. Bright was with the United States Engineer Department, Nashville District, on the improvement of the Cumberland River, making location surveys for Locks Nos. 2, 3, and 4. He also acted as Inspector in local charge of the construction of Lock No. 3.

From February to July, 1896, he was engaged as Superintendent for the contractors on the construction of a masonry wall enclosing the Tennessee State Penitentiary and also superintended the construction of a 4-mile extension of the Nashville, Chattanooga, and St. Louis Railway, for the Duck River Phosphate Company.

In July, 1896, he again entered the service of the U. S. Engineer Department, serving until December of that year as Overseer of the Nashville District; from January, 1897, to June, 1898, as Draftsman and Photographer in the office at Nashville; and from June, 1898, to

* Memoir prepared by B. M. Taylor, Asst. Valuation Engr., Louisville and Nashville Railroad, Louisville, Ky.

March, 1900, in local charge of the construction of Lock No. 6 on the Cumberland River. In May, 1900, Mr. Bright was transferred to the Mobile District and put in local charge of the construction of Lock No. 5 on the Warrior River, Alabama, remaining on this work until August, 1901.

At this time, he concluded to exploit his engineering talent in another field, and, for a number of years, was engaged in railroad location and construction on the Tennessee Central Railroad, the Louisville and Nashville Railroad, the Tidewater Railway, and the Nashville, Chattanooga, and St. Louis Railway. On the Tennessee Central Railroad, he was employed as Resident Engineer on construction and Assistant Engineer on maintenance of way. On the Louisville and Nashville Railroad, he was Resident Engineer at Dossett, Tenn., for 2 years, constructing several miles of road, including the Dossett Tunnel. He was then employed for about 14 months on the Tidewater Railway, part of the time on location surveys and part as Resident Engineer on construction. His last railroad work was on various surveys for the Nashville, Chattanooga, and St. Louis Railway, covering about two years prior to April, 1908.

The service of the Government again attracted Mr. Bright, and, in April, 1908, he took charge in the field of the completion of Colbert Shoals Canal and Lock on the Tennessee River at Riverton, Ala., remaining there about five years. On this project he had charge of extensive concrete river wall construction, the grading of the canal trunk, the erection of the massive lock-gates and their maneuvering gear, and the construction of steamboats, barges, and dredges after the canal was opened to traffic.

Malaria contracted at Riverton so affected his health, that he had to give up river work. He then removed to Nashville, Tenn., where, from August, 1913, to August, 1914, under the direction of the City Engineer, he rebuilt that portion of the City Water Reservoir, which shortly before had failed destroying considerable property in its vicinity.

In August, 1914, Mr. Bright became a member of the Engineering Staff of the Division of Valuation, Interstate Commerce Commission, serving as Assistant Field Engineer until June, 1917, and as Field Engineer from that date until his death, which occurred at his home in Nashville, Tenn., on April 3d, 1919.

Mr. Bright was married on May 8th, 1898, to Miss Laura Doxey, of Hendersonville, Tenn.

Throughout his career, he was always loyal to his profession, conscientious and competent in the performance of his duties, and, in his death, the Engineering Profession lost one of its best men.

Mr. Bright was elected an Associate Member of the American Society of Civil Engineers on December 3d, 1902. He had also been a member of the Engineering Association of the South since 1890.

ROBERT EDWARD DAKIN, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 15TH, 1918.

Robert Edward Dakin was born at Gaylordsville, Conn., on July 2d, 1888. He was graduated from the New Milford High School in 1906, and three years later received the degree of Bachelor of Science (Civil Engineer) from the Sheffield Scientific School of Yale University. In the fall of 1909, following his graduation from Yale, Mr. Dakin entered the service of the New York, New Haven, and Hartford Railroad, in charge of electric railway construction.

Mr. Charles S. Mellen, then President of the New Haven Railroad, in an endeavor to carry out his great plan of a unified transportation system for New England, had acquired for feeders to the steam and water lines an extensive system of trolleys, and, until April, 1917, Mr. Dakin was engaged in the reconstruction of many of these properties, the construction of extensions and connecting links, and, in the latter half of the period, in the preparation of maps, land records, mileage and tax reports, appraisals, and other forms of data, for submission to the various commissions having jurisdiction or interest in various features of the property.

In detail may be mentioned surveys and estimates for the construction of about 20 miles of new trolley line and for the relocation of several smaller sections; the surveys and estimates for an 8-mile, wood pole, transmission line and for an 11-mile, steel tower line, including the construction of the former; the construction of car-houses at Derby and New Haven; of freight-houses at Ansonia, Naugatuck, New Haven, and Waterbury; the enlargement of the Bridgeport steam station; the improvements and additions to the Bulls Bridge hydro-electric plant, the latter work, although not of great magnitude, called for a high degree of ability and ingenuity, as the new penstock and stand-pipe with the concrete pier in the tail-race, and the new head and forebay gates, had to be installed with practically no interference with the operation of the plant, the operating head of 108 ft. making the tail-race work which came just at the ends of the draft tubes, particularly difficult; the construction of a reinforced highway bridge over the penstocks and the widening of the canal for several hundred feet, also had their difficulties, but to a less degree. Of Mr. Dakin's office work may be mentioned the design and estimates for a 4 000-kw., 40-ft. head, hydro-electric plant, and appraisals of the power plants, car-houses, and other buildings and structures of the trolley line between New London and the Massachusetts State Line, of the entire property

* Memoir prepared by Charles Rufus Harte, M. Am. Soc. C. E.

of the Westport Water Company, and of many smaller groups of trolley properties of The Connecticut Company.

Successively filling positions of steadily increasing responsibility, Mr. Dakin's work came to the attention of the interests which had just purchased the Housatonic Power Company for the New York, New Haven, and Hartford Railroad, and, in April, 1917, he went with them, successfully handling the purchase of much right of way and the adjustment of changes of highways and bridges, necessary for the construction of a 75-ft. head, 24 000-kw. capacity, hydro-electric plant on the Housatonic River, at Stevenson, Conn. Later, when construction started, Mr. Dakin was transferred to the J. A. P. Crisfield Contracting Company, and, as Assistant Engineer, had charge of all surveys, maps, and highway changes, the latter involving a number of country roads and some State highways. In connection with the latter, Mr. Dakin constructed over the Pomperaug River a reinforced concrete cantilever bridge of arch form, with a central span of 90 ft., and semi-arches 45 ft. long on either side, the design being the joint work of Mr. Dakin and R. L. Saunders, Deputy State Highway Commissioner of Connecticut. The pool, 10 miles in length, fills a narrow valley with steep hills and deep intersecting ravines on each side, making surveys difficult and the determination of the economic relocation of the disturbed highways a problem calling for engineering ability of high order, while the natural difficulties of the field work were greatly increased by the fact that most of it had to be done during the severe winter of 1917-18.

Mr. Dakin was completing this work, and had organized the forces and was directing the surveys of other power and storage projects, when, after a very brief illness, he died at Danbury, Conn., on December 15th, 1918, a victim of the influenza epidemic of that winter. On September 13th, 1913, he was married to Marion Evans, of Gaylordsville, Conn., who, with a son, Theodore, born on November 11th, 1916, survives him. Another son, Edward, born on January 28th, 1918, and Mrs. Dakin's mother died within a few hours of Mr. Dakin.

"Rob" Dakin, as he was best known to his friends, was a man of engaging personality, combining to a rare degree technical ability, executive capacity, and a "sunshiny" disposition next to impossible to ruffle. Although cut down at little more than the beginning of a career of great promise, he left no small monument in good work well done, and in the hearts of his friends an undying memory.

Mr. Dakin was elected a Junior of the American Society of Civil Engineers on December 2d, 1914, and an Associate Member on January 15th, 1917.

STANLEY DEAN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1918.

Stanley Dean, the son of John Dean and Elizabeth Stevenson Findlaw Dean, was born in Manchester, England, on February 22d, 1879.

At the age of nine he came to the United States with his parents, his father being engaged in engineering contracting at that time. His education was obtained in various schools, mostly in the South and West, as his father's business necessitated moving from place to place.

In 1901 he entered Washington University, St. Louis, Mo., where he remained two years. The family then settling in Chicago, Ill., Stanley entered the Civil Engineering Department of the Armour Institute of Technology, from which he was graduated in 1905 and received the degree of Civil Engineer in 1910.

The following record of Mr. Dean's professional experience is taken from his application for a commission in the Quartermaster Corps, U. S. Army:

1900-05, Timekeeper, Material Clerk, Carpenter, and Concrete Foreman, successively, during all of 1900 and to September, 1901; during the summers of 1902-03-04, on construction of concrete arches, culverts, abutments, piers, and earthwork incidental to double-tracking the Grand Trunk Western Railway. In 1905, Superintendent of Construction, for John Dean and Company, of Sixth Avenue Arch Bridge, Des Moines, Iowa. In December of the same year superintended the construction of abutments and culvert at Muir, Mich., for the Grand Trunk Western Railway. In 1906, in the employ of Hugh L. Cooper, M. Am. Soc. C. E., as Draftsman and Designer on parts of McCall Ferry and Keokuk water-power developments, made surveys and gaugings of Trent River, at Healeys Falls, Ontario, and was Transitman on topographical surveys for water-power developments at Schaghticoke and Johnsonville, N. Y.; in charge of reconnaissance work for water power on Connecticut River and East Canada Lakes, New York, and Engineer in Charge of location, surveys, and borings for foundations, etc., at Schaghticoke, N. Y. In 1907, Engineer in Charge of preliminary surveys and investigations, Ingham Mills water-power development, East Canada Creek, New York, and made a similar survey and investigation on Broad River near Columbia, S. C.; April, 1907, engaged on coffer-dam and stone cribwork investigation for proposed 125-ft. concrete dam foundations, East Creek, New York; May, 1907, engaged on Broad River surveys, South Carolina; June-August, 1907, East Creek investigations; September-December, 1907, Resident Engineer

* Memoir prepared by Alfred E. Phillips, M. Am. Soc. C. E.

in charge of field engineering work, assisting Superintendent of Construction in development of water power at Schaghticoke, N. Y., 20 000 h. p., 150-ft. head. This work involved building an ogee gravity dam averaging 25 ft. high, 700 ft. long, in solid concrete, together with intakes, canal, ½ mile long, 120 000 cu. yd. of earth and rock excavation, 12-ft. riveted steel pipe line, 800 ft. long, steel surge-tank, penstocks, and power-house, 56 by 110 ft., for four units, the cost of the work being $1 250 000. In 1908 was engaged on above described work. January-April, 1909, in a similar position, finishing water-power development at Johnsonville, N. Y., 4 000 h. p.; May-August, 1909, Superintendent of Construction on water-power development, Fountain Falls, Montreal River, Ontario, until abandonment of work due to consolidation of companies; then Engineer on construction of Ragged Chutes transmission lines, and spiral riveted pipe carrying compressed air to Cobalt Mining camp; made report on Matabitchouan River development. All the foregoing water-power work was done in the employ of Viele, Blackwell, and Buck, of New York City.

In 1910, Mr. Dean was appointed Instructor in Civil Engineering at the Armour Institute of Technology, having charge of instruction in hydraulics, water power, reinforced concrete, and general construction methods and costs. In 1912 he was made Assistant Professor of Civil Engineering, and, in 1917, Associate Professor of Hydraulic Engineering.

During the summer of 1912 Professor Dean was engaged as Resident Engineer for Solomon, Norcross, and Keis, at Watervliet, N. Y., on the construction of a 100-ft. earth dam for the storm-sewer system. During the summer of 1913 he was associated with the writer in the design and supervision of construction of a hollow concrete dam on a sand foundation, at Hamlin Lake, Michigan.

From June to October, 1917, Professor Dean was Principal Assistant to the Supervising Engineer, Department of Constructing Quartermaster, at Camp Jackson, Columbia, S. C., in charge of all surveys and layout work for topography, roads, railroad track and yards, buildings, sewers, water supply, etc.; and, from July to October, he was in charge of the engineering design and supervision of construction of water lines, reservoir of reinforced concrete having a capacity of 1 250 000 gal., pumping station, 30 miles of sewers, septic tank, and other incidental structures. Professor Dean returned to the Armour Institute of Technology in October, 1917, remaining until January, 1918, when he resigned to become Principal Assistant to the Supervising Engineer, Department of Constructing Quartermaster, Camp A. A. Humphreys, Accotink, Va., in charge of railroad location, general surveys, and dock construction.

He received a Captain's commission on August 24th, 1918, and was then made Supervising Engineer in the Constructing Quartermaster Corps at Camp Humphreys, Va. He was supervising the building of a dam and power plant at Springfield, Va., in connection with Camp Humphreys, when, on October 2d, 1918, he was instantly killed at a railroad crossing.

He was in an automobile on his way to the dam at Springfield. A colored chauffeur was with him, but Capt. Dean was driving at the time. The crossing at this place is very dangerous, the view of the tracks being obstructed. He had waited for a train to pass and then started across the tracks only to be caught by a fast train from the opposite direction. The chauffeur jumped and escaped without injury, but Capt. Dean, in trying to save himself and the car, stalled the engine and then jumped to save himself. He was struck and thrown 100 ft., and instantly killed.

As a student, instructor, engineer, and a man among men, Stanley Dean was one in a thousand. Absolutely honest, upright, clean mentally and morally, he commanded the love and respect of all with whom he came in contact. He possessed a keen, analytical mind, infinite patience, and ability of concentration that was reflected in whatever he undertook. He loved to teach, and his professional experience, combined with up-to-date knowledge of his specialty, made him an ideal instructor.

He was married on December 14th, 1913, to Miss Ellen Hallaway, of Schaghticoke, N. Y., who, with a daughter, survives him.

Capt. Dean was elected an Associate Member of the American Society of Civil Engineers on April 1st, 1908.

JOHN HULL DECKER, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 10TH, 1918.

John Hull Decker was born in Sioux City, Iowa, on September 25th, 1878. The family moved to Brooklyn, N. Y., where Mr. Decker attended the public schools, and was graduated from the Manual Training High School in 1898. During the summers of 1895, 1896, and 1897 he was employed in the shops of the Thomson Meter Company, of Brooklyn, assisting in experiments on water meters, and, in 1898, he took up the study of surveying in the office of Mr. Charles Crook, of Brooklyn, and afterward with Mr. R. L. Williams, of the same place, first, as Rodman and then as Instrumentman. In January, 1900, he entered the employ of the late Richard Lamb, M. Am. Soc. C. E., of New York City, and was in responsible charge of topographical surveys for electric railways, water mains, sewers, and road construction, and assisted in the design and construction of a copper mining plant.

Throughout 1901, Mr. Decker was with the Babcock and Wilcox Company, of Bayonne, N. J., as Draftsman on water-tube boilers, Inspector on the erection of factory buildings, and Engineer, Inspector, and Surveyor on switchyard work.

In January, 1902, he entered the employ of the Atlantic City Sewerage Company, Atlantic City, N. J., as Engineer on construction for three months and then as Superintendent and Engineer of the company, having responsible charge of the design, construction, and maintenance of the sewerage system, involving re-designing, enlarging, and reconstructing the system, comprising about 50 miles of pipe sewers.

From January, 1906, until his death, Mr. Decker was Engineer and General Manager of the Atlantic Construction and Supply Company, of Atlantic City, engaged in general contracting, particularly of sewerage systems, water-works, highway construction, and reinforced concrete work. His work as Contractor in the erection of the new electric light power plant at Atlantic City attracted favorable attention. He also built the new Somers Point-Longport Boulevard. Probably his most notable work, apart from that of rebuilding the Atlantic City sewerage system, was done in connection with T. Chalkley Hatton, M. Am. Soc. C. E., on the design and construction of the Atlantic City drainage canal.

Mr. Decker was a self-made man and engineer, well known all over South Jersey and in many other communities. He was affable and

* Memoir prepared by F. Herbert Snow, M. Am. Soc. C. E.

whole-souled, and his friendship was valued. Although cut off in the prime of life, and at a time when the business for which he had struggled had become well established, and with bright prospects ahead, he left an estate that will provide comfortably for his widow and child.

Mr. Decker was elected a Junior of the American Society of Civil Engineers on September 6th, 1904, and an Associate Member on December 5th, 1906.

GUSTAVO ADOLFO DUBOIS, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1918.

Gustavo Adolfo Dubois was born at Matanzas, Cuba, on September 25th, 1882. He was a graduate of the Massachusetts Institute of Technology, having received, in June, 1908, his degree of B. S. in Civil Engineering.

After his graduation, Mr. Dubois returned to Cuba and entered the employ of the Public Works Department of the City of Havana, as Assistant Engineer in the Department of Civil Construction. In October, 1909, he was appointed Resident Engineer in charge of water-works construction at Cienfuegos, Cuba. As such, he had immediate supervision over the construction of the 10 000 000-gal. reinforced concrete reservoir for the storage of the city water supply, work representing an outlay of $240 000, and also directed the survey and design of various other structures in connection with the new water-works.

From Cienfuegos, Mr. Dubois was called to Havana as Assistant Engineer in charge of sewer and drain construction under D. E. McComb, M. Am. Soc. C. E., then Chief Engineer of the Department of Sewers and Paving of the City of Havana, starting on his new duties in September, 1910. He was closely associated with the work of designing and constructing the sewerage system of the entire city, the cost of which amounted to about $15 000 000. He remained with the Department for three years, until September, 1913, when, the work of construction being nearly finished, he accepted, at the special request of the late Dr. Enrique Nuñez, then Secretary of Health of the Republic of Cuba, an appointment as Chief Engineer of the Sanitary Department of the Republic.

Mr. Dubois' early connections with sanitary projects and construction, and his specialization in Sanitary Engineering while at college, well fitted him for his new responsibilities, and it was a credit to his efficiency and executive ability that, in spite of several political changes, he was able to win the confidence of the successive Administrations under which he was called to serve. During his connection with the Sanitary Department of Cuba, he was influential in helping to maintain the high degree of sanitary standards which have placed the largest cities of the Island, Havana especially, among the most healthful in the world.

In February, 1918, Mr. Dubois resigned his position with the Sanitary Department, to accept the management of one of the leading industrial concerns of Havana, the "Tropical" Brewery, pertaining to the Nueva Fábrica de Hielo, a company owning and operating the

* Memoir prepared by G. A. Hernández, Esq., Havana, Cuba.

two largest breweries on the Island. He had hardly started on his new duties when, after a short illness, he died on August 30th, 1918.

Mr. Dubois was one of the leaders of the young generation of Cuban engineers who, educated at the leading American universities, have contributed so much toward the development of Cuba's resources, making this little Island one of the most prosperous in the world. His early death, at the time when success was beginning to crown his untiring efforts, will be regretted by all who had had personal contact with him and could appreciate the sterling qualities of his character.

Mr. Dubois was married in 1909, to Miss Rita M. Lavastida, of Matanzas, Cuba, who, with a daughter and his mother, survives him.

He was a Member of the Cuban Society of Engineers, and also held the degree of Civil Engineer from the University of Havana.

Mr. Dubois was elected a Junior of the American Society of Civil Engineers on September 1st, 1908, and an Associate Member on May 6th, 1914.

CLARENCE FREDERICK EBERLY, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 18TH, 1918.

Clarence Frederick Eberly was born at Perrysburg, Ohio, on November 30th, 1882. He received his early education in the grade and high schools of Perrysburg, and, on his graduation from Ohio State University in June, 1906, entered the service of the United States Government as Junior Topographer in the United States Geological Survey. He gave a course of engineering instruction at the Harvard Engineering Camp, in the summers of 1909, 1910, and 1911, which gained for him the praise of Hector J. Hughes, M. Am. Soc. C. E., the Director of the camp. He remained in the Survey Service until November, 1912. During Mr. Eberly's connection with that department he had charge of various work, including the mapping of 225 sq. miles in West Virginia, making topographic surveys in Idaho and Arizona, the making of a map of Seattle for the Alaska-Yukon and Pacific Exposition, besides considerable office work which was done at the headquarters of the Survey, in Washington, D. C.

Mr. Eberly left the Survey Service in November, 1912, to help take charge of the establishing of a geographical survey department for the Argentine Republic, but only spent one year in Argentine, returning in December, 1913, to the United States Geological Survey.

Mr. Eberly left the Survey again in April, 1914, to engage in a real estate and building proposition as Secretary of The Heffelbower Realty Company, of Toledo, Ohio. After successfully putting through this venture, he joined the staff of the Lucas County Surveyor, in May, 1917, as Senior Engineer, in charge of highway surveys and construction, but, in July, 1918, obtained a leave of absence in order to render what aid he could in the construction of the Government Air-Nitrates Plant, then building in Toledo. On this work he had charge of the construction of streets and highways, and, by his untiring efforts and ability, rendered valuable service. He left the Nitrates Plant in September in order that he might be able to donate his services to the Government as Instructor of Field Engineering at the Toledo University for the Student Army Training Corps, and was engaged in this work at the time of his death. While instructing his class he was stricken with an attack of influenza which in three days developed into pneumonia, from which he died in less than one week, on December 18th, 1918.

He was unmarried, and leaves, besides his father and mother, Mr. and Mrs. F. C. Eberly, of Perrysburg, Ohio, two brothers: Somers L.

* Memoir prepared by Cecil L. Rood, Esq.

Eberly, of Toledo, and Capt. Karl C. Eberly, who served with the American Expeditionary Forces in France.

Mr. Eberly was a member of Phœnix Lodge, No. 123, F. and A. M., a member of the Kappa Sigma Fraternity, and was elected an Associate Member of the American Society of Civil Engineers on September 6th, 1910.

WILLIAM WETMORE GIBBS, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1919.

William Wetmore Gibbs was born in Melbourne, Fla., on January 12th, 1889. From his earliest years he had a marked taste for constructive work and applied science, and this led him to take up the study of Civil Engineering, in spite of a great many difficulties. He entered the University of Florida in 1905, worked his way through by his own energy and industry, and was graduated in 1909 with the degree of B. S. C. E., having been the first graduate in Civil Engineering from that University. His record as a student was one of brilliant achievement in his studies, combined with prominent activity and leadership in student enterprises. After his graduation he always retained a hearty interest in the University of Florida, frequently visiting it and keeping up his acquaintance among its faculty and alumni.

Immediately after his graduation Mr. Gibbs was placed in charge of a part of the work of planning and laying out the suburb of Jacksonville known as Ortega. A few months later he entered the employment of The Phosphate Mining Company, at Nicholas, in Polk County, Florida, where, on account of his splendid ability, and notwithstanding his youth, he was rapidly advanced through various positions until he became Assistant General Manager of the company and second in charge of this large enterprise.

Immediately on the entry of this country into the World War, he gave up his position, went to Washington for training, volunteered for service in France, was commissioned First Lieutenant, and sent overseas, being attached to the 6th Engineers, Company B. After several months' service, his health broke down and he was invalided home for recuperation. Much to his distress, he was unable again to satisfy the medical officers of his physical fitness for further active service, and, after a few months, he was given an honorable discharge from the Army. Pending his discharge, Lieut. Gibbs threw himself actively into the educational campaigns for Liberty Bonds and Red Cross funds, and, fresh from his experiences in France, was able to speak with very great effectiveness in these campaigns. Immediately on his discharge from the Army, he opened an office as Consulting Engineer at Gainesville, Fla., and quickly developed a successful practice. While engaged on some of his professional work near Green Cove Springs, early in January, 1919, he contracted influenza, from which he died scarcely more than a week later in Jacksonville.

On November 2d, 1910, Mr. Gibbs was married to Miss Lucia Patton, who, with three children, survives him.

* Memoir prepared by Dr. J. R. Benton, Secretary, Florida Engineering Society.

He was not only a man of great native ability and high promise in his profession, but also possessed unusually fine personal qualities. He had to an extraordinary degree the faculty of winning both the good will and the confidence of all with whom he came in contact. It was his temperament to follow with a great deal of enthusiasm whatever he undertook; and this spirit was characteristic of his attitude to his chosen profession, in which he took both pride and pleasure, and for the higher ideals of which he displayed the greatest devotion.

Mr. Gibbs was a charter member of the Florida Engineering Society. In 1916 the University of Florida granted him the degree of C. E. He was a member of the Alpha Tau Omega fraternity and of the Phi Kappa Phi honorary fraternity.

Mr. Gibbs was elected a Junior of the American Society of Civil Engineers on October 7th, 1914, and an Associate Member on May 15th, 1917.

NORMAN MARSHALL HALCOMBE, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1919.

Norman Marshall Halcombe, the son of the late Arthur Follett Halcombe, of Fern Grove, Urenui, Taranaki, New Zealand, was born at The Pines, Feilding, New Zealand, on July 29th, 1878.

He attended the public schools in New Zealand, and in August, 1905, he entered Stanford University as a special student. He was graduated, with the degree of B. A. in Civil Engineering, in 1909. While at Stanford University he was engaged in numerous activities, was President of the Stanford Society of Civil Engineers, a prominent member of the Eucina Club, and took part in several dramatic productions.

Mr. Halcombe served as an Apprentice in the Machine Works of David Murray and Company, at Wanganui, New Zealand, from March, 1899, to October, 1900, and then, until November, 1901, as a Machinist (Improver) at the Machine Works of Joseph Sparrow and Son, at Dunedin, New Zealand, on the construction and erection of gold dredges. He served as Foreman and then as Draftsman with Goniuan and Company, Engineers, of Newcastle, New South Wales, from March, 1902, until March, 1905, on colliery plant and machinery. His next work was with the Schäake Machine Works, of New Westminster, B. C., Canada, as a Draftsman on the design of cannery and saw-mill machinery.

He was employed as Draftsman for the Bay Cities Water Company from February, 1906, until July, 1907, on computations, etc., for the Sierra Water Supply for San Francisco, and was then engaged, until August, 1908, as Field Draftsman on the survey of the Yuba and Feather Rivers, and as Instrumentman and in charge of level party for the California Débris Commission. From the latter date until November, 1908, Mr. Halcombe was Chief Draftsman for the Stone Canyon Coal Company, and was engaged in designing head-works, tracks, trestles, bridges, buildings, tipple structure, dam, etc. Then, until September, 1909, he was Contractor's Engineer for the Atlantic, Gulf, and Pacific Company, of San Francisco, in charge of dredging Oakland Harbor, dredging for wharf foundations for Government docks at Fort Mason, San Francisco, and the construction of three bulkhead structures in Oakland Harbor. From September, 1909, until May, 1910, he was Junior Engineer with the City of San Francisco, on the construction of a section of the overflow conduit. He then became Engineer of the Carmen Creek Mining Company, of Salmon, Idaho, and was engaged in the erection of an experimental gold mill.

* Memoir prepared by E. W. Sawyer, Assoc. M. Am. Soc. C. E.

Mr. Halcombe served for five years as Chief Engineer with Messrs. Duryea, Haehl, and Gilman, on large irrigation projects, surveys, compilations of data, computations, design, construction, etc. He was then engaged in private practice in San Francisco for two years.

His action with regard to the World War was very praiseworthy, as he gave up a lucrative practice in San Francisco, and traveled to London at his own expense to offer his services in assisting to defeat the Germans. He joined the Royal Engineers early in 1915, and was commissioned as Second Lieutenant on May 1st of that year. He was promoted to Lieutenant a month later, and gained his Captaincy on August 4th, 1915. He commanded and trained the 31st Cheshire Field Company, R. E., from June 1st to August 20th, 1915, and was then ordered to Gallipoli as Captain with the 21st Welsh Field Company, R. E. At Gallipoli he was transferred and placed in command of the 21st Cheshire Field Company, R. E., and, later, was sent to Egypt on special evacuation work, followed by special transport work in the Western Desert, Egypt and Sinai, for which he was mentioned in dispatches.

Capt. Halcombe joined the Royal Flying Corps on November 1st, 1916, and organized and developed the Engine Repair Depots, in Egypt and Palestine. He did some special work and reported on technical subjects in connection with the Air Service. He received his commission as Major on November 16th, 1917, and was twice mentioned in dispatches.

Maj. Halcombe returned to England on September 1st, 1918, in connection with the liaison work of the Royal Flying Corps then being carried out in the United States. Arriving in London at a time when technical men were scarce, his contemplated trip to America was forestalled, and he was attached for immediate service to the Technical Department, Parachute Section, of Aircraft Production.

At the time the Armistice was signed he was still in this position, as Major in the Royal Engineers and the Royal Air Force. He expected to return to San Francisco to re-open his business there in connection with the firm of Halcombe, Flanders, and Read, but was offered an important staff position on the Indian Establishment of the Air Force in India, and made arrangements to sail for India, on January 23d, 1919. On the day before he sailed, he was married at St. Mary's, Battersea, to Miss Blanche Stuart, eldest daughter of Wesley and Amy Turton, of "Hawkshead", Lake Wakatipu, New Zealand. Mrs. Halcombe followed her husband on the next boat. He was attacked with influenza and pneumonia on the voyage, and at Port Said was removed from the steamer to the Red Cross Hospital in a very weak condition from which he did not recover, but passed away on February 15th, 1919.

Maj. Halcombe is survived by Mrs. Halcombe, who has returned to her home in New Zealand; a brother, A. H. Halcombe; and two sisters, Mrs. M. A. Taylor and Mrs. M. L. Gordon Glassford.

"Joe", as he was familiarly called by his associates, was a great favorite, especially among his engineering friends. He had a quick wit and a thorough appreciation of a joke. He had a high conception of the duties of an engineer, and as he was absolutely thorough in everything he did, his judgment in engineering as well as in personal matters was relied on by all who were in close touch with him. His sacrifice in the war is a monument to his conscientiousness, and the example he has set will not soon be forgotten. He will be sincerely missed by his many friends and by all who were privileged to know him.

He was a valuable member of the Press Club, of San Francisco, and, on leaving in 1915 to join the English forces, his fellow-members made him an Honorary Member.

Maj. Halcombe was a member of the Western Society of Engineers, and of several clubs in London. He was elected a Junior of the American Society of Civil Engineers on September 3d, 1907, and an Associate Member on May 2d, 1911.

ROBERT DWIGGINS MONTEITH HENLEY, Assoc. M. Am. Soc. C. E.*

DIED MAY 3D, 1918.

Robert Dwiggins Monteith Henley was born near Richmond, Ind., on February 15th, 1879.

During his summer vacations of 1898 and 1899, Mr. Henley was employed as Rodman with the Cleveland, Cincinnati, Chicago, and St. Louis Railway Company.

In October, 1899, he entered the employ of the Louisville and Nashville Railroad Company, as Draftsman in the Mechanical Department. He remained with this Company until November, 1904, serving as Draftsman in the Chief Engineer's Office, Assistant to the Resident Engineer, Resident Engineer, and Assistant Engineer on the Pensacola, the Pensacola and Atlantic, the Birmingham, Selma, and New Orleans, and the Southern Alabama Divisions, of that Company.

From December, 1904, to May, 1905, Mr. Henley was a student at the Central University of Kentucky, but returned to the Louisville and Nashville Railroad Company as Draftsman in the Chief Engineer's office. During September and October, 1905, he was employed as Draftsman in the Bridge Engineer's office of the Cleveland, Cincinnati, Chicago, and St. Louis Railway Company.

In October, 1905, Mr. Henley entered the service of the Chicago, Rock Island, and Pacific Railway Company, and, until April, 1906, was engaged as Resident Engineer for the Morris Terminal Railway Company. From April to June of the same year, he served as Assistant Engineer in charge of the construction of the Morris Terminal Railway and the Kaw River Bridge, at Kansas City, Mo., and, from June, 1906, to February, 1907, he was employed as Assistant Engineer in charge of the Rose Hill Coal Spur near Oskaloosa, Iowa. After the contractors had finished their work, Mr. Henley had charge of the organization of the force to move about 80 000 cu. yd. of material, of which about 30 000 cu. yd. was classified. From March to October, 1907, he served as Transitman on location and as Estimator for proposed grade reduction work for the Railway Company.

In October, 1907, Mr. Henley left the service of the Chicago, Rock Island, and Pacific Railway Company to enter Earlham College, Richmond, Ind., as a special student, where he remained until May, 1908. At that time, he was employed as Building Superintendent for the Murry Sun Amusement Company, in charge of the construction of the Murry Theater at Richmond, Ind.

In August, 1908, he entered the employ of the Atchison, Topeka, and Santa Fé Railway Company as Building Inspector, and also as Designer of reinforced concrete work, with headquarters at Topeka,

* Memoir prepared from information on file at the Headquarters of the Society.

Kans. Mr. Henley retained this position until 1912, when he went to the Kansas City Southern Railway Company. While with this Company he was connected with the work of the Chief Engineer's office, at Kansas City, Mo.

In 1913, Mr. Henley accepted a position as Assistant Engineer with the Central Coal and Coke Company of Kansas City, Mo., which position he held at the time of his death which occurred on May 3d, 1918. While with this Company, he was engaged in the design and supervision of various improvements around its mines, yards, and saw-mills.

He was a comparatively young man, and with his wonderfully brilliant mind and his wide experience in the various branches of engineering, he no doubt would have attained an eminent position in the Engineering Profession, had he lived.

Mr. Henley was elected a Junior of the American Society of Civil Engineers on September 3d, 1901, and an Associate Member on January 31st, 1911.

BRUCE RITCHIE HONEYMAN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 5TH, 1918.

Bruce Ritchie Honeyman was born at Portland, Ore., on December 30th, 1885, of Scotch parentage. He received his early schooling at the Portland Academy, from which he went to the Massachusetts Institute of Technology and was graduated from there in 1906.

On his return to Portland, Mr. Honeyman was employed as Structural Engineer with Morris H. Whitehouse, Architect, and remained with the firm for some years. He then went with the Hurley-Mason Company as Structural Engineer, designing and superintending the construction of several buildings. He left this Company to go to work in Vancouver, B. C., Canada, for the Norton-Griffiths Company. After the completion of his work there, Mr. Honeyman went to St. Paul, Minn., and, for eleven years, was employed as Engineer by the F. J. Romer Construction Company. At the time of his entry into the military service, he held the position of Manager of General Contracting and Construction for the Company.

On May 11th, 1917, Mr. Honeyman entered the military training camp, at Fort Snelling, Minn., as a civilian candidate for a commission in the Engineers. He was transferred later to the Central Training Camp for Engineers, at Fort Leavenworth, Kansas, where he was assigned for instruction to the 13th Provisional Training Regiment. He served with this regiment as a candidate until August 14th, 1917, when he was graduated. On August 15th, 1917, he was commissioned as Captain of Engineers, United States Army, and assigned to duty at Camp Dodge, Iowa, with the 313th Engineers, in command of Company A.

Capt. Honeyman proceeded overseas with his regiment during the early part of September, 1918, and served with it until September 28th, 1918, at which time he was confined to quarters on account of sickness which later developed into pneumonia, resulting in his death on October 5th, 1918. Maj.-Gen. W. M. Black, M. Am. Soc. C. E., Chief of Engineers, states concerning Capt. Honeyman's service with the United States Army: "His services were satisfactory in every respect," and it would seem to those who knew him that these words express the opinion of his work throughout his entire career.

In June, 1909, he was married to Marion Blakely, of Tacoma, Wash. He is survived by a daughter, Elinor, and a son, Blakely.

Capt. Honeyman was elected an Associate Member of the American Society of Civil Engineers on September 5th, 1911.

* Memoir prepared by George C. Mason, M. Am. Soc. C. E.

PAUL ROBERT KIRSTEIN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 17TH, 1918.

Paul Robert Kirstein was born at Cincinnati, Ohio, on April 20th, 1888. He finished his preparatory course for college in the Technical School of the University of Cincinnati in 1905, and, in 1909, he was graduated from Purdue University, receiving the Bachelor of Science Degree in Civil Engineering.

Mr. Kirstein's first work was with the United States Coast and Geodetic Survey as Topographical Draftsman, which position he retained for three months.

In 1909, he entered the Engineering Department of the City of Cincinnati, where he remained until 1916. His first work was with the Highway Division and, later, with the Sewer Division. In 1911, he was transferred to the Division of Structures where he was engaged on the design of various small works such as concrete steps, culverts, bridges, etc. In 1912, he was appointed Inspector of reinforced concrete work on the Gilbert Avenue Viaduct, a structure costing about $250 000. During 1913 and 1914, Mr. Kirstein was employed as Assistant Resident Engineer on the Ludlow Avenue Viaduct, a reinforced concrete structure costing about $350 000, and in 1915 and 1916, he served as Resident Engineer on the Hopple Street Viaduct, a reinforced concrete structure costing about $430 000. After finishing this work, he was engaged on the design of various concrete structures, the principal one of which was the McMillan Street Bridge, estimated to cost about $150 000.

When the work of planning the proposed rapid transit system for the City of Cincinnati was commenced in 1917, Mr. Kirstein was appointed Designing Engineer for the Board of Rapid Transit Commissioners. His work in this connection was on the design of the various structural features such as subway, reinforced concrete elevated structure, trestles, etc. He held this position at the time of his death.

Mr. Kirstein was a very capable and energetic young man, of high character and pleasing personality. His untimely death closed a promising career and was a distinct loss to the Engineering Department which he served.

He was married, on September 26th, 1914, to Miss Freda Kohstall, of St. Bernard, Ohio, who, with a son, survives him.

Mr. Kirstein was elected an Associate Member of the American Society of Civil Engineers on May 15th, 1917.

* Memoir prepared by Frank L. Raschig, M. Am. Soc. C. E.

ANGUS ROBERT MACKAY, Assoc. M. Am. Soc. C. E.*

DIED JUNE 29TH, 1918.

Angus Robert Mackay, the eldest son of the late Senator Robert Mackay, was born at Montreal, Que., Canada, on October 27th, 1872. After his preparatory education, he entered the Massachusetts Institute of Technology, from which he was graduated in Mining Engineering in 1894. During 1894 and 1895, he studied at the Berg Academie and the University of Berlin, Germany.

On his return to the United States in 1895, Mr. Mackay was employed by the Horseshoe Mining and Milling Company, of Deadwood, S. Dak., as Manager of its mining interests and of its reduction plant, the Kildonan Mill, at Pluma, S. Dak. He retained this position until the property was sold in 1901.

From 1902 to 1904 he was engaged in research work and study for mining interests at Sault Ste. Marie, Ontario and Michigan, and also at Sydney, Nova Scotia.

In 1904 Mr. Mackay joined the Engineering Staff of the Electrical Development Company of Ontario, at Niagara Falls, Ont., Canada, on one of its large hydro-electrical projects. He also had charge during this time of some hydraulic surveys and investigations near Lake Superior for Hugh L. Cooper, M. Am. Soc. C. E.

From 1906 to 1907 he was identified with, and had an interest in, a foundry enterprise near Montreal, Que., Canada; and, from 1907 to 1908 was engaged as Advertising Manager for the Canadian Fairbanks Company, with headquarters at Montreal.

In 1908 Mr. Mackay entered the employ of the General Exploration Company, of Boston, Mass., and was sent by that Company to Wickenburg, Ariz., as Manager of the Vulture Mine. He supervised the construction of a 20-stamp mill and a cyanide plant, and developed the property into a successful business. He retained this position until his death, which occurred suddenly at Oakland, Cal., on June 29th, 1918.

Mr. Mackay's activities were numerous and varied; he was particularly interested in the development of the natural resources of Arizona. To his friends and business associates, who were many, his death came as a shock, and he will be greatly missed by all who knew him and worked with him. He had been a member of the American Institute of Mining Engineers since 1896.

Mr. Mackay was elected an Associate Member of the American Society of Civil Engineers on October 29th, 1912.

* Memoir prepared from information on file at the Headquarters of the Society.

HUNTER McCLURE, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 26TH, 1918.

Hunter McClure, the son of Hunter H. McClure and Alice Camp McClure, was born in Rome, Ga., on December 23d, 1887.

After his preparatory education, he entered the Georgia School of Technology from which he was graduated in June, 1908, with the degree of B. S. in Civil Engineering. He then entered Cornell University and because of his previous engineering experience during his vacations and his advanced credits, he was granted the degree of Civil Engineer in June, 1910.

From September, 1909, to September, 1910, Mr. McClure was engaged on railroad work as Transitman on valuation surveys on the lines of the Central of Georgia Railway Company in Alabama; Chainman and Field Draftsman for the Oregon Short Line Railroad Company on preliminary surveys and location in Utah, Oregon, and Idaho; and as Inspector on the construction of reinforced concrete snowsheds in the Cascade Mountains, for the Great Northern Railway Company.

In September, 1910, he returned to Cornell University as Instructor in the Mechanics of Engineering at Sibley College; he also did postgraduate work in railway, bridge, and hydraulic engineering.

From June, 1911, to February, 1912, Mr. McClure was in the employ of the American Bridge Company as Draftsman on shop detail drawings of steelwork for mill and office buildings, schools, hotels, etc., having been stationed at the Company's Pencoyd and Brooklyn plants and in the New York office.

In March, 1912, he returned to the West, where he spent two months in the offices of the Northern Pacific Railway Company, at St. Paul, Minn., and Tacoma, Wash. He then entered the service of the Idaho Northern Railway Company, and was employed until March, 1913, as Instrumentman and Inspector on the construction of the Payette River Bridge.

In March, 1913, Mr. McClure returned to the American Bridge Company as Draftsman at its Gary Plant, but remained only until July, when he was appointed Draftsman in the Bridge Department of the Chesapeake and Ohio Railway Company, and was engaged on bridge design, the design of roadway structures, plans for the construction of tidewater coal piers, etc.

He remained with this company until October, 1914, when he accepted a position with the Division of Valuation, Interstate Commerce Commission, and was sent to the Pacific District. After a month or so in the San Francisco office, Mr. McClure was made Assistant Field Engineer in charge of a field party, which position he

* Memoir prepared from information on file at the Headquarters of the Society.

resigned in December, 1917, to enlist in the Reserve Engineer Corps, then being organized for service in the World War.

He was commissioned a First Lieutenant and trained at Camp Lee, Va. He was afterward assigned to Company N, 21st Engineers, and spent some time at Camp A. A. Humphreys, Va., from which place he was sent with his regiment to France in 1918. After his arrival in France, he was taken suddenly ill, with spinal meningitis on September 25th, 1918, and died the following day.

Lieut. McClure was elected a Junior of the American Society of Civil Engineers on January 4th, 1910, and an Associate Member on October 9th, 1917.

WILLIAM MENKE, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 1ST, 1918.

William Menke, the son of John and Louise Sommers Menke, was born at New York City, on December 2d, 1885. His early education was received in the public schools of New York, and, in 1904, he entered Columbia University, from which he was graduated with the degree of Civil Engineer in 1908.

Immediately after his graduation, Mr. Menke was employed as Draftsman and Transitman, on the construction of a 2 000-h. p., hydro-electric plant at Walden, N. Y.

In February, 1909, he entered the employ of the New York Board of Water Supply as Engineering Inspector in the Reservoir Department, in connection with the construction of the Catskill Aqueduct. After about one year as Inspector, at Brown's Station, Mr. Menke was appointed an Assistant Engineer with the Water Board, and transferred to the Southern Aqueduct Department near White Plains, N. Y. While in this Department he was in charge of the field work on the construction of the concrete cut-and-cover aqueduct, steel pipe siphon, and gate-chambers, at Scarsdale, N. Y. He was engaged for about two years in this work. He was then transferred to the City Aqueduct Department, in the Manhattan Division, and was employed on the construction of a 48-in., cast-iron main and, subsequently, on two shafts and the tunnel from 23d Street to 42d Street, New York City.

In August, 1914, Mr. Menke left the Water Board and received an appointment as Engineer with the New York Public Service Commission, First District, on subway construction in New York City. He was in the Commission's employ for two years as Assistant to the Section Engineer, having had charge of the field parties and office work on the construction of the Lexington Avenue line between 43d and 53d Streets. The work on this section, valued at more than $2,000,000, consisted of a four-track, steel and reinforced concrete structure, which was built partly in open cut and partly in tunnel. To the regret of his associates, he resigned in August, 1916, to accept a position as Draftsman and Designer with the Bethlehem Steel and Bridge Company, at its plant in Steelton, Pa.

Mr. Menke's next move carried him back to New York City, where, in June, 1917, he entered the employ of the J. G. White Engineering Corporation. For the first few months, he was engaged as Designer and Draftsman in the New York office, and was then sent to Kingston, N. Y., where he was Assistant to the Resident Engineer and, later, Resident Engineer, on a 50-mile, 60 000-volt, transmission line.

* Memoir prepared by Ralph E. Drake, Assoc. M. Am. Soc. C. E.

In June, 1918, Mr. Menke accepted a position as Field Engineer with the Construction Department of The Babcock and Wilcox Company, at its plant in Barberton, Ohio. Activities here consisted of alterations and improvements to existing buildings, the erection of a new machine shop, the installation of electric cranes and other machinery, and improvements and reconstruction work on plant railroad lines, craneways, etc. Mr. Menke's duties were arduous, but he always performed them with a willing and cheerful spirit. This, his last, position as Engineer on industrial plant construction, was one for which he felt he was particularly fitted and in which he was best satisfied to work.

Mr. Menke's work was always well and carefully done, and he was held in high esteem by his numerous associates, both for his valuable, painstaking, faithful services and his pleasing personality. His quiet, unassuming nature and his consideration for others endeared him to all with whom he came in contact.

He died on November 1st, 1918, at the Citizens' Hospital, Barberton, Ohio, a victim of influenza followed by pneumonia, his death having occurred after an illness of one week. He was unmarried.

Mr. Menke was elected a Junior of the American Society of Civil Engineers on December 6th, 1910, and an Associate Member on April 16th, 1918.

OGDEN MERRILL, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 5TH, 1918.

Ogden Merrill was born in the old Williamsburg section of Brooklyn, N. Y., on December 9th, 1877. His father was one of the founders of Merrill Brothers, which firm was originally located at Grand Street, New York City, as a ship chandlery, in the old days when sailing vessels were still common along the wharves on the East Side. This firm is still in existence, but is now engaged in the manufacture of drop forgings. Mr. Merrill's father is still alive, hale and hearty at 89 years.

Mr. Merrill received his education at the Boys High School, Brooklyn, and at Cornell University, from which he was graduated in the course of Civil Engineering, in 1899.

After graduation, Mr. Merrill entered the employ of the late Ira A. Shaler, M. Am. Soc. C. E., acting as an Assistant Engineer and Superintendent on the construction of the rock tunnel under Park Avenue, New York City, for the Rapid Transit Commission, during the construction of which Mr. Shaler lost his life. Mr. Shaler was noted for his policy of surrounding himself with young engineers, who received their early training under his supervision. Many of those who were graduated from this practical school are now engaged in the construction business for themselves.

After the completion of the foregoing work, Mr. Merrill was employed by the United Engineering and Contracting Company as Engineer and Superintendent, in charge of the construction of the electric pumping station for the dry docks of the New York Navy Yard, the placing of the deck on the elevated structure of the Atlantic Avenue Improvement, Brooklyn, and the bridge over Croton Lake for the Putnam Division of the New York Central and Hudson River Railroad.

Later, Mr. Merrill was Superintendent on the construction of the North River Tunnels of the Pennsylvania Railroad, in New York City, and completed the East River Tunnels of the New York Subway system under Mr. T. J. Brown.

In 1908, Mr. Merrill associated himself with Mr. Albert Ruckgaber in the formation of the Merrill-Ruckgaber Company, Constructing Engineers. Under Mr. Merrill's supervision, as President, this Company has been engaged on numerous projects, confining itself principally to heavy masonry construction and pneumatic work, among the more important of which are the following: Depressed boiler house for the Westchester County Court House at White Plains, N. Y.; pneumatic foundations for the Assay Office, New York City; concrete bridges for the Delaware, Lackawanna and Western Railroad, the Long Island Railroad, and the Central Railroad of New Jersey;

* Memoir prepared by Albert Ruckgaber, Esq.

concrete coal pockets for the Central Railroad of New Jersey; water supply system for the City of Cumberland, Md., including a large concrete dam and pipe line; and pneumatic pier foundations for bridges at Chattanooga, Tenn., and Lawrence, Mass.

Mr. Merrill was the inventor of a method of sinking piers and caissons by the use of a system of nested shaft liners, adapted to be expanded telescopically downward by jacking, as the excavation proceeds. This method he patented and used successfully in sinking caissons for the foundations of the Assay Office Building.

Mr. Merrill left for his summer place at Mountainville, N. Y., on September 27th, 1918, in good health. He apparently carried with him the germs of influenza, which, soon after his arrival at Mountainville, developed into a very virulent form, finally resulting in pneumonia. He died on October 5th, after an illness of only six days.

He was married in 1903 to Miss Laura Ruckgaber, of Brooklyn, N. Y., who, with two children, survives him.

He was noted, among those who knew him best, for his novel and ingenious methods of construction, based on sound engineering principles and carried out by him with marked success. He had a very practical strain, and was an authority on all questions relating to sub-surface and sub-aqueous construction. His upright character and lovable disposition, together with his policy of dealing fairly with men in all walks of life, endeared him, not only to a host of friends, but to those who served under him.

Mr. Merrill was elected an Associate Member of the American Society of Civil Engineers on February 3d, 1904.

ADELBERT PHILO MILLS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 20TH, 1918.

Adelbert Philo Mills was born in Tecumseh, Mich., on November 10th, 1883. His parents were Mr. and Mrs. Adelbert Wortman Mills. He was educated in the public schools of Tecumseh, having been graduated from the High School in 1902. He entered the University of Michigan the same year, and was graduated in 1906, with the degree of Bachelor of Science in Civil Engineering; he received the advanced degree of Master of Science in 1909.

From May, 1906, until May, 1907, Mr. Mills was an Assistant Instructor in the University of Michigan, and also Assistant in the office of C. G. Wrentmore, M. Am. Soc. C. E., in Ann Arbor, Mich., for whom he designed, estimated, and inspected the construction of several buildings in Ann Arbor, and a reinforced concrete factory building in Jackson, Mich. He also had field charge of the survey of eighty city lots.

From May, 1907, until October, 1908, Mr. Mills was Assistant Engineer of the American Bottle Company, of Newark, Ohio, and was employed on the construction of new glass plants at Newark, and at Streator, Ill. These plants included a reinforced concrete warehouse and power-house, a large factory building, glass furnaces, gas producers, water, gas, and steam piping, electric wiring, conveying equipment, etc., and, on a large part of this work, he was in responsible charge.

From October, 1908, until October, 1909, Mr. Mills was an Assistant Instructor and took a post-graduate course in the University of Michigan, and then, until June, 1910, was Acting Assistant Professor of Civil Engineering in the College of Civil Engineering of Cornell University. In June, 1910, he was appointed Assistant Professor of Materials at Cornell, which position he held in May, 1918, at the time he received his commission as Captain of Engineers, U. S. Army.

From May to December, 1911, he also had responsible charge of the testing of materials used in the construction of Potter's Falls Dam for developing a new water supply for Ithaca, N. Y.

Professor Mills left Ithaca in May, 1918, and was ordered first to Camp Lee, Va. Later, he was transferred to Camp Oglethorpe, Ga., where he remained in training until the latter part of September. He sailed for France on September 29th, and reached there about October 14th, at which time he cabled his family, informing them of his safe arrival. They heard nothing further until November 26th when a message from the War Department announced his death from cerebro-spinal meningitis, in a hospital in Brest, on October 20th, 1918.

* Memoir prepared by E. E. Haskell, M. Am. Soc. C. E.

Professor Mills is survived by his wife and two children, Elizabeth and Adelbert. Mrs. Mills spent the summer of 1918 with her husband at Camp Oglethorpe, and is at present with her parents in Ypsilanti, Mich.

In the death of Professor Mills, Cornell College of Civil Engineering suffers a serious loss. As a teacher, he stood for lofty ideals and thorough work. His own sense of duty was so high, and his scorn for inaction so great, that he was intolerant of laziness and indifference, so that his classes were animated by an intense spirit of energy.

His personal interest in investigation, fortified by his manual skill in devising and constructing apparatus, as well as by his connection with actual problems of engineering practice, gave advanced students in his Department a field of view and a practical interest that proved of great value educationally and reacted in the greater reputation of the College.

In the Faculty, he was invariably a champion of high standards and good work. Impatient of excuses and intolerant of carelessness, he aimed at the establishment of a greater reputation for the College through the better training of its graduates, and though this attitude sometimes savored of harshness, and though students sometimes complained of the rigor of his requirements, he kept always before himself his ideal and refused to compromise with inadequate preparation or inferior ability.

Professor Mills' greatest achievement in scholarship was the production of a thoroughly up-to-date book entitled "Materials of Construction," published in 1915. It has been adopted extensively as a textbook by engineering colleges, and as a standard reference work by engineers in practice. It secured his recognition as an authority on this subject.

Professor Mills was elected a Junior of the American Society of Civil Engineers on September 3d, 1907, and an Associate Member on September 3d, 1913.

JAMES POWELL MURRAY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 28TH, 1918.

James Powell Murray, son of James W. and Juliette Wingo Murray, was born in Dallas, Tex., on January 18th, 1880. When he was eight years of age his parents removed to Coleman, Tex., where he attended the public schools. He was graduated from the Coleman High School in 1896, and, in 1897, he entered the University of Texas, at Austin, remaining until June, 1901. Mr. Murray returned to the University in 1902, and was graduated with the degree of Bachelor of Science, in June, 1903. Late in the session of 1903-04, he again returned to the University, and did special work throughout that year and at various other times when he had the opportunity, and, finally, was graduated with the degree of Civil Engineer, in June, 1918.

Mr. Murray taught school in Coleman County, Tex., in 1896 and again in 1902. In the summer of 1903 he accepted employment in the United States Geological Survey as an Assistant in a topographic mapping corps in charge of Arthur Alvord Stiles, M. Am. Soc. C. E., Topographer, then engaged in mapping the Chisos Mountains, in Brewster County, Texas. At that time this was a wild section of the Texas-Mexican frontier, known as the "Big Bend" country. The main camp of the topographers was in a canyon in the heart of the mountains, separated from civilization and the railroad by an arid waste of cactus and rocks, a hundred miles or more by any route a man might take. In these solitudes Mr. Murray began his professional career, and learned, from the severe surroundings, the practical things that seem to have served him best in the years that followed. He helped in the geodetic location of Glenn Springs and Boquillas, two places which years later came into National notice over night, when Mexican bandits raided the Big Bend and killed American soldiers guarding the border.

Mr. Murray spent 1905 and 1906 in the Philippine Islands, in the United States Civil Service, as an Assistant Civil Engineer for the Board of Port Works. Returning from the Philippines, he was employed as an Engineer with the Santa Fé Railway on work in Eastern Texas, where he remained until December, 1907. From 1909 to 1913 he was Assistant Engineer of Austin, Tex., and did extensive work in street improvement.

In 1913 Mr. Murray entered the State Reclamation Department of Texas, as Principal Assistant and Civil Engineer, his time being about equally divided between field and office work and service in an advisory capacity. He ran many miles of geodetic traversing and primary leveling, the results of which are published in *Bulletin No. 7* of the State Reclamation Department. He made hydraulic surveys, examined the

* Memoir prepared by Arthur A. Stiles, M. Am. Soc. C. E.

engineering plans of levee districts, and frequently represented the State Reclamation Engineers in matters coming before the Courts. He made the computations for *Bulletins Nos. 5, 5A, and 6,* of the State Reclamation Department, and was joint author of *Bulletin No. 8* of the same Department, entitled "Factors for the Interconversion of Latitudes and Departures, in Feet, and Differences of Latitude and Longitude, in Seconds, 1918."

On June 27th, 1918, Mr. Murray entered the service of the United States as a Captain of Engineers, and went to Camp Lee, Va., for training. While there his company was transferred to Camp A. A. Humphreys, where he completed his course of study. On September 12th, he was ordered to report for duty to the Chief of Engineers at Washington, D. C. He was then ordered to his home in Austin on furlough and to await further orders. At the training camp or *en route* home he contracted Spanish influenza which rapidly developed into pneumonia, and, about four days after arriving at his home, he died. He was buried in Oakwood Cemetery, Austin, with military honors extended by the officers and men of Camp Mabry.

Capt. Murray was married to Miss Alberta Blackburn at Austin, Tex., on April 29th, 1907. His wife and a son, Edward Stuart Murray, survive him.

He was a man of high ideals, a competent civil engineer, and a cultured gentleman, esteemed by the public, and trusted and beloved by his associates.

Capt. Murray was elected an Associate Member of the American Society of Civil Engineers on November 28th, 1916.

ALLEN EUGENE NICHOLS, Assoc. M. Am. Soc. C. E.*

DIED MAY 8TH, 1919.

Allen Eugene Nichols, the son of Eugene M. and Ellen M. Nichols, was born at Madison, Wis., on September 30th, 1888. When he was six years old, his parents moved to Philadelphia, Pa., his father being engaged in the engineering and contracting business. His education, however, was obtained mostly in the West.

In 1906, he entered the Civil Engineering Department of Purdue University, Lafayette, Ind., from which he was graduated in 1910. In 1913, he received the degree of Civil Engineer from the same institution.

On his graduation from college, Mr. Nichols entered the employ of the Baltimore and Ohio Railroad Company, with which company he remained until December, 1910, when he resigned to go with R. L. Sackett, M. Am. Soc. C. E., Consulting Engineer for the State of Indiana, as Assistant in the design of water-works and sewage disposal plants. From April, 1911, to May, 1912, Mr. Nichols was employed by the Chicago and Western Indiana Railroad Company as Assistant Engineer on track elevation and maintenance work. From May, 1912, to February, 1914, he was associated with Alvord and Burdick, Consulting Engineers, as Assistant Engineer on the design and construction of sewage disposal and water supply plants.

From February, 1914, to May, 1915, Mr. Nichols was a student at the University of Pennsylvania, where he made a special study of the chemistry of water. From May, 1915, to January, 1918, he was employed as Engineer in direct charge of the design, specifications and contracts for the Bureau of Waste Disposal of the City of Chicago, Ill. During this time, Mr. Nichols made reports for the Department of Public Works on the mechanical and chemical character of garbage tankage produced by the various cities of the United States, and established a comparison with the standard requirements of the fertilizer trade. He also took charge of the design and specifications for the water purification plant of Oshkosh, Wis.

When the United States entered the World War, Mr. Nichols endeavored to enlist in the Army, but owing to a permanent injury, he was not accepted by the military authorities. He, however, entered the employ of the Du Pont Engineering Company, as Engineer of Construction for a 70 000 000-gal. water filtration plant for the Government powder factory at Old Hickory Works, Nashville, Tenn., which position he retained until construction was completed.

On February 1st, 1919, Mr. Nichols, with his father and brother, entered into a general engineering and contracting business. For many years previous he had given special study to the subject of the disposal

* Memoir prepared by T. McLean Jasper, Assoc. M. Am. Soc. C. E.

of sewage and municipal waste and the utilization and reduction of garbage.

"Nick", as he was called by his immediate friends, was one of those quiet, solid fellows, retiring to the casual individual, but to those to whom he gave his friendship, there was no reserve. Those who had enjoyed that friendship miss the man he was and appreciate the richness of the qualities he had shared among them in the relatively short life he lived.

Mr. Nichols died at his home in Chicago, Ill., on May 8th, 1919, after a short illness, of pneumonia, a transfusion of blood to a near relative a few months before his death, having left him with less than normal vitality to fight the disease.

Mr. Nichols was an Associate Member of the American Society of Mechanical Engineers, a Member of the Franklin Institute, and a Member of the American Water-Works Association and the Sons of the American Revolution.

Mr. Nichols was elected an Associate Member of the American Society of Civil Engineers on October 10th, 1916.

WILLIAM ARTHUR O'BRIEN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 12TH, 1918.

William Arthur O'Brien, a son of Judge E. J. O'Brien, prominent in the legal life of St. Louis, Mo., was born in that city on October 5th, 1886. After acquiring a common school education, he entered Christian Brothers College, from which he was graduated in the Engineering Course.

He was usefully employed during his vacations, and following his graduation, he entered the service of the Merchants Bridge Terminal Railroad Company as a Rodman. In August, 1905, he was engaged as Instrumentman with the Chicago, Peoria and St. Louis Railway, and remained with that road until November 1st, 1906, when he was appointed an Assistant Instructor in Railroad Engineering at Purdue University, where, at the same time, he took a special course in mathematics.

In the early part of 1908 Mr. O'Brien was employed as Assistant Engineer for the Little River Drainage District, with headquarters in Cape Girardeau, Mo., and at once became an active participant in the planning of this, the largest drainage district actually undertaken in America, containing about 1 000 sq. miles. On December 1st, 1909, he returned to the Chicago, Peoria and St. Louis Railway, as Engineer of Maintenance of Way, in complete charge of the track and the Bridge and Building Department. There he found a field in which his executive ability manifested itself.

In 1910 the Chief Engineer of the Little River Drainage District, Mr. Otto Kochtitzky, resigned, and the position was tendered to Mr. O'Brien. He accepted it and, on December 1st, 1910, entered upon the performance of the onerous duties connected with that position, discharging them with marked ability. His work there called attention to him, and caused him to receive attractive offers of employment, all of which he declined.

Mr. O'Brien served as a member of the Board of Engineers who prepared to give expert testimony in connection with the litigation arising out of the filing of the plan for the Miami Conservancy District. He was the youngest member of that Board of distinguished engineers.

His powers of analysis were remarkable, and he was so well grounded in mathematics that his knowledge was readily at his command.

In 1912 Mr. O'Brien was married to Miss Amee Conkling, of a prominent family of Springfield, Ill. She survives him, and mourns a very devoted husband.

* Memoir prepared by the late Isham Randolph, M. Am. Soc. C. E.

Naturally high-strung and forceful, he kept his impulses under control and developed a judicial temperament, in which justice moulded firmness. Thus he was in life's business. In life's friendships he was true and generous, and those who were associated with him loved him for all the admirable traits they recognized in him.

Mr. O'Brien was elected an Associate Member of the American Society of Civil Engineers on January 2d, 1912.

HAROLD COE OGDEN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1918.

Harold Coe Ogden was born in Buena Vista, Colo., on June 12th, 1886. After two years' study in the University of Colorado, he entered active engineering work in 1906. For three years he was engaged principally in mining work in Nevada. From 1909 to 1913 he was with the Arkansas Valley Sugar Beet and Irrigated Land Company at Holly, Colo., on hydrographic work, surveys, and construction.

Coupled with his engineering qualifications, which were of a high character, Mr. Ogden possessed unusual business ability, with the faculty of applying his engineering knowledge of methods and efficiency to practical business problems. He entered commercial life in 1913 as Assistant Manager of the Seattle plant of the Pacific Coast Biscuit Company. He was unqualifiedly successful in his business career, and, at the time of his death, was President and Treasurer of the Seattle Baking Company, which is the largest concern of its kind in Seattle. He was also State Chairman of the Bakers' Service Commission.

Mr. Ogden possessed a pleasing and genial personality which made and held for him a wide circle of friends who are deeply grieved over his early going.

Although for the past few years Mr. Ogden had not been in the active practice of his profession, he continued to have a lively interest in engineering matters, and looked forward to the time when he would resume his engineering activities.

The epidemic of influenza was responsible for Mr. Ogden's death, in Seattle, Wash., on October 22d, 1918. He is survived by a widow and three daughters.

Mr. Ogden was elected an Associate Member of the American Society of Civil Engineers on October 29th, 1912.

* Memoir prepared by S. G. Porter, M. Am. Soc. C. E.

EDWIN JAMES POTTER, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 18TH, 1918.

Edwin James Potter, the son of Charles F. and Lenora E. (Sayles) Potter, was born in Bridgeton, R. I., on March 17th, 1884. He attended the graded schools of Burrillville, R. I., and was graduated from the Moses Brown School, of Providence, R. I. He took the civil engineering course at Brown University, and received the degree of Bachelor of Science in Civil Engineering in 1907.

During the summer of 1906, Mr. Potter was an Assistant in the office of the City Engineer of Providence, and in June and July, 1907, he was engaged on surveys and drawings for a mile of Macadam road for Burrillville.

In August, 1907, he entered the employ of the B. F. Smith Construction Company, Mill Builders and General Contractors, of Pawtucket, R. I., and for several years thereafter was engaged in estimating, drafting, laying out, and superintending the construction of mills, power plants, storehouses, etc., in New England and New York State.

Mr. Potter's interest in matters relating to engineering was keen, and he lost no opportunity to increase his knowledge of his profession. With this object, he visited the Canal Zone, in order to inspect the Panama Canal before its completion, as well as many other engineering works.

In October, 1913, he took charge of the Hopkins Machine Works, in Bridgeton, and, under his management, the company prospered and increased its business. This concern had been owned by Mr. Potter's family for generations. It was founded by Augustus Hopkins, his great-grandfather in 1836, and from him it went to Mr. Potter's great-uncle, Horatio Hopkins. His grandfather, James Potter, was connected with the company, and his father, also.

Mr. Potter took a keen interest in everything that would be beneficial to the town, and could be relied on to lend a helping hand in any movement for its betterment.

He was a member and Secretary of Pascoag Hose Company No. 2, Past-President of the Alumni Association of the Burrillville High School and of the Moses Brown Alumni Association, and a member of Sigma Chi of Brown University. In all patriotic affairs in this district, he was an active worker, having served as a member of the Liberty Loan Committee and as Chairman of the Burrillville Committee in the War Savings Stamps drive.

* Memoir prepared from information on file at the Headquarters of the Society.

Mr. Potter was a Republican, but did not take an active part in political affairs until 1918, when, after considerable urging, he consented to accept the nomination for State Senator.

He was very much interested in the welfare of the Free Baptist Church, of Pascoag, R. I., and, although he had never united with the church, he had contributed liberally toward its support.

He contracted influenza and died, after a two weeks' illness, at the home of his parents. His death was a shock to the community in which his life had been spent, and a distinct loss to the town. Besides his parents, he is survived by a sister, Mrs. Charles L. Osler, of Malden, Mass., who, shortly after her brother's death, moved to Bridgeton, R. I.

Mr. Potter was elected a Junior of the American Society of Civil Engineers on October 1st, 1907, and an Associate Member on October 1st, 1913.

CAREY SIMON PRATT, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 25TH, 1918.

Carey Simon Pratt was born in Quincy, Ohio, on July 7th, 1868. Some time during his young manhood he decided to become a Civil Engineer and make that Profession his life work. Accordingly, from 1888 to 1891, he acquired his technical training in the Ohio State University. Almost immediately after finishing his college course he began serving the public in an official capacity as Deputy County Surveyor in Noble County, Indiana, and continued for the greater part of his life in the public service, although for certain periods he engaged in private work as a Civil Engineer.

From 1893 until 1895, Mr. Pratt served as City Engineer of Sidney, Ohio, and for three years thereafter engaged in private practice as an Engineering Contractor, later, becoming Resident Engineer for the Dayton-Springfield and Urbana Railway. From 1900 to 1902 he occupied the position of Chief Engineer of the Springfield-Xenia Electric Railroad, and, at the same time, acted as Consulting Engineer for the Western Ohio Electric Railroad. It is obvious that he was particularly active during these few years in the development, organization, and maintenance of some of the most important electric railway enterprises in that section of the country and at a time when the electric railroad was being developed at a rapid rate. Men of initiative, ingenuity, training, and ability were just as much required in those days as they were in the early history of steam railroading, and it can be truthfully said that Mr. Pratt possessed all these characteristics in a marked degree.

In 1902, he again returned to public life as City Engineer of Urbana, Ohio, and continued in that position for about eleven years. While serving in this capacity, he designed the sanitary sewer system of Urbana, embracing some fourteen miles of sewers. He also acted as County Engineer of Champaign County, Ohio, from 1902 to 1909, specializing particularly on the design and construction of highway bridges.

From 1913 to 1916 Mr. Pratt was engaged in county bridge work at Sandusky, Ohio, and was also associated with Smith and Boulay of Toledo, Ohio, Consulting Engineers, at both of which places his wide training and the experience gained in his municipal work were of the utmost advantage.

During 1916 and 1917, he supervised one of the first pieces of brick highway construction in West Virginia, just outside of Morgantown. On the completion of that project he came east in the spring of 1917

* Memoir prepared by Harry F. Harris, Assoc. M. Am. Soc. C. E.

and entered the employ of Mercer County, New Jersey, in the capacity of Bridge Engineer. He obtained the appointment to this position through a competitive examination held by the New Jersey State Civil Service Commission, having attained the highest rating of all the competitors for that position. It will be noted that his very decided liking for municipal and public work again asserted itself. Mr. Pratt served as Bridge Engineer until his untimely death.

He was always an intensive student. In fact, it can be said that his real studies did not begin until after he left college. The acquirement of technical knowledge in all the branches of engineering with which he had been connected was one of his greatest hobbies. For years he made a practice of collecting all articles describing any unusual, noteworthy, or ingenious methods for carrying out difficult construction problems, formulas, and new developments pertaining either to the technical or practical side of bridge and highway construction, and as a result of his activities in this direction he had accumulated a rather interesting collection of data and engineering literature which he greatly prized. It was this intense and constant study with so little relaxation that hastened his death.

It is indeed unfortunate that the lives of so many men should be taken just at a period when their age and experience should render them of the utmost usefulness to those they serve, and this means not only the community and mankind generally, but also their families and friends as well. Such was the case when Carey Simon Pratt died on November 25th, 1918.

In 1891 Mr. Pratt was married to Ada Belle Rogers, of Columbus, Ohio. As a result of this union three children—Mrs. Harold Houston, Helen and Lindsay Pratt—were born.

Mr. Pratt was connected with the Masonic Fraternity, in which he took a deep and ardent interest. He was a member of Champaign Lodge, Free and Accepted Masons, of Urbana, Ohio, and also a Knight Templar, belonging to Rapier Commandery of the same city. He was also a member of the American Society for the Advancement of Science.

Mr. Pratt was elected an Associate Member of the American Society of Civil Engineers on June 4th, 1913.

WARD HALL REAM, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 4TH, 1918.

Ward Hall Ream, son of Mr. and Mrs. J. J. Ream, was born in Oshkosh, Wis., on October 15th, 1886. He attended the public schools of that city, and was graduated from its High School in 1905. In the fall of that year he entered the Webb Academy of Marine Engineering and Naval Architecture, in New York City, from which he was graduated in June, 1908, and then took a post-graduate course at Columbia University in advanced bridge designing and reinforced concrete.

His first work was with the American Bridge Company. He then served as Bridge Draftsman with the Public Service Commission. In 1910 he entered the employ of the Delaware, Lackawanna and Western Railroad Company as Designer and Estimator of Bridges and Stations. His work included the design of railroad and highway bridges, trestles, towers, and various other forms of construction. It was during this period that the railroad did such a large quantity of improvement work, and, with the study of its problems in steel construction, Mr. Ream was intimately connected. He always took especial pride in his work in connection with the construction of the Martin's Creek and Tunkhannock Viaducts on the Clark Summit-Halstead Cut-Off.

In 1915, Mr. Ream joined Company C of the 22d Regiment of Engineers, N. G. N. Y., under the command of Col. Vanderbilt, and, in the summer of 1916, went with his Company to the Mexican border. On his return he proceeded with his work on the Delaware, Lackawanna and Western Railroad improvements, until the United States entered the World War. He then entered the Officers' Training School at Camp Myers, Va., and was awarded a Captain's commission. On account of his age, he would not be allowed to take command in active service, and he thereupon asked for a rerating to First Lieutenant. This was granted, and he was assigned to Company C, 305th Engineers. He went overseas in May, 1918.

On October 2d he took charge of his company, relieving Capt. Matthew J. Hoff, and his recommendation for promotion to a captaincy was forwarded to headquarters. In the early morning of October 4th, his company was ordered forward to assist the infantry in an attack. Lieut. Ream had his company in place, and was just finishing his round of inspection to see that everything was all right, when a high explosive shell struck near him. A fragment hit him in the back of the neck, killing him instantly. He was buried, with military honors, in a little concrete vault which was constructed near the spot where he fell.

His admirable qualities made him popular, not only with his intimate friends, but with all those with whom he came in contact in a business

* Memoir prepared by Edward W. Wendell, Assoc. M. Am. Soc. C. E.

and military way. He was terse, modest, persevering, and courageous, and possessed that other indefinable attrib te which carries with its possessor the ability to see through the veneer of pretence into the heart of the real. He was a most lovable and loyal friend, and his friends mourn his departure, but rejoice in the fact that when his Maker called, he went with the highest honor that a soldier can attain: "Killed in Action."

Besides his parents, he is survived by a sister, Miss Vinnie Ream, of Oshkosh, and one brother, Edward L. Ream, of St. Paul, Minn.

Mr. Ream was elected an Associate Member of the American Society of Civil Engineers on April 18th, 1916.

ALEXANDER CHARLES SCHILLING, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 4TH, 1919.

Alexander Charles Schilling was born at New York City, on October 20th, 1886. He acquired his early education in, and was graduated from, the public schools of that city. Later, he received private instruction, and also studied at the Morris High School and the College of the City of New York.

From October, 1904, to July, 1917, with the exception of a short period (October, 1905, to February, 1906), when he was engaged as Transitman with the Rapid Transit Railroad Commission of New York City, Mr. Schilling was employed in the office of the President of the Borough of the Bronx, New York City. In this position, he was engaged chiefly in the construction of sewers and roads.

In July, 1917, he was appointed Assistant Engineer, and, later, Chief of Sewers, United States War Department, and, in this capacity, he designed and constructed sewage works, including septic tanks and disposal plants, located and constructed buildings, theatre, barracks, etc., and supervised the construction of concrete roads, water supply, etc., at Camp Meade, Md.

Of Mr. Schilling's work at Camp Meade, Leo Hudson, Assoc. M. Am. Soc. C. E., who was associated with him there, writes:

"Mr. Schilling came to Camp Meade, Md., from New York City, and proved to be a tireless worker and a man of great worth. He was ambitious and strove continually to improve his own condition and the conditions of those with whom he was associated. His coming to Camp Meade was a personal and professional sacrifice, brought about through a high sense of patriotic duty to help his country in its time of need. He did important work while at Camp Meade, and was there practically from the beginning to the ending of the work. He left behind a good record and a spirit of good fellowship. He left Camp Meade to undertake other work for the Government, which work paid salaries less than those he could have secured from his former employers. He was a splendid young man, and his death is a loss to both the country and the Profession."

On December 19th, 1917, Mr. Schilling was appointed Field Assistant with the U. S. Geological Survey, and had charge of a party controlling areas for topographic mapping. Later, he was detailed in connection with military mapping and served in this capacity until he left the service, on January 15th, 1919.

Mr. Schilling was taken ill while on his way to Mobile, Ala., where he had accepted a position in the U. S. District Engineer's Office. He

* Memoir prepared from information on file at the Headquarters of the Society.

died in Mobile, on February 4th, 1919, from influenza, followed by broncho-pneumonia. His death came as a great shock to his family and to all his associates.

He was married on September 24th, 1910, to Miss Grace Mattock Willey, of New York City, who, with two children, Alexander Ralph and Grace Marjorie, survives him.

Mr. Schilling had a keen appreciation of Nature, and was a lover of outdoor life. His boyhood days were spent largely in the woods, and he was greatly interested in ornithology, having had, at one time, a large collection of live birds. He was a vigorous young man, and no amount of work phased him. All his leisure time, and most of his working hours, were spent in the open. Several years ago, he was the possessor of an auxiliary sloop yacht, and his knowledge of the channels, together with his excellent eyesight, made him a capable captain. He had explored the country by motorcycle and automobile, and was extremely fond of travel, quoting many, many times:

> "To him who in the love of Nature holds
> Communion with her visible forms, she speaks
> A various language;"

Mr. Schilling was elected an Associate Member of the American Society of Civil Engineers on June 18th, 1918.

GEORGE CORLISS SEE, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 28TH, 1918.

George Corliss See, son of James W. and Hester Rose See, was born in Hamilton, Ohio, on June 25th, 1882. He received his preliminary education in the public schools of Hamilton, and was graduated from the Rensselaer Polytechnic Institute in 1905, with the degree of Civil Engineer.

Early in life Mr. See evinced an aptitude for the profession he was to follow. In 1900 he entered the office of his father, Mr. J. W. See, Consulting Engineer, in the capacity of a Draftsman and Designer. Prior to entering the Rensselaer Polytechnic Institute, he was employed as a Rodman by the United States Engineer Department, in Havana, Cuba. Upon graduation Mr. See was employed by the Manhattan Trap Rock Company, at Nyack, N. Y., as Assistant Superintendent, until October, 1905, when he entered the employ of the New York and Long Island Railroad Company. His thorough and painstaking work while connected with the latter company earned for him rapid promotion. In 1906 he had risen from the grade of Rodman to that of Chief of Party. In this capacity Mr. See had charge of the grade and alignment of the Manhattan end of the Belmont Tunnels.

In October, 1906, he became Assistant Engineer of the Fonda, Johnstown and Gloversville Railroad Company. In addition to the duties of general maintenance-of-way work of the road, Mr. See supervised the construction of a four-arch concrete bridge for a grade crossing elimination. He also designed and supervised the construction of many other improvements undertaken by the company during his two years stay with this road.

The last ten years of Mr. See's life were spent in highway work. He was Assistant Engineer and Assistant to the Division Engineer, New York State Highway Department, with headquarters in Albany, from March, 1908, to August, 1917. After his first few years in this service, Mr. See became recognized as one of the foremost highway engineers in his Department. He was entrusted with the supervision of all highway construction in five counties of the Division. In May, 1914, he secured a leave of absence from the State Department of Highways, and, forming a partnership with Harrison Smith, Assoc. M. Am. Soc. C. E., opened an office in Temple, Tex., for the practice of highway engineering. The firm did an extensive business, and it was due to failing health that Mr. See relinquished the field and returned to his former duties in Albany.

In August, 1917, Mr. See was appointed Senior Highway Engineer for the United States Department of Public Roads, with headquarters

* Memoir prepared by J. C. Watts, Engr., Harbor and Dock Comm., Troy, N. Y.

in Montgomery, Ala. A few weeks prior to his death, he had received orders for transfer from Montgomery to Washington, D. C. While spending a short vacation in Troy, N. Y., before proceeding to his headquarters, he was taken ill, and died the second day following.

Besides his wife and son, Mr. See is survived by a host of friends. His genial nature made him loved by all who knew him. His careful, efficient, and painstaking manner made him always a valuable asset to whatever corporation he served. His brilliant and scholarly attainments have made for him a record well worthy of his profession.

Mr. See was a member of Sigma Xi fraternity and of the Rensselaer Society of Engineers. He was elected a Junior of the American Society of Civil Engineers on September 3d, 1907, and an Associate Member on September 2d, 1914.

ROBERT EARLE SHIPLEY, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 11TH, 1918.

Robert Earle Shipley, the son of Dr. John L. and Mary E. Shipley, was born on December 30th, 1887, at Cullychoha, Indian Territory. His parents died when he was a small boy, and he made his home with his aunt, Mrs. Sarah McConnell. After finishing the public schools, he entered the High School in his home town, and, later, the University of Arkansas, from which he was graduated with the degree of B. C. E., in June, 1911. He was a member of the Phi Kappa Alpha Fraternity.

Mr. Shipley's professional career was as follows: March to August, 1909, Field Draftsman, with the Arkansas Co-operative Constructing Company, Bentonville, Ark., on preliminary and location surveys; July to December, 1911, Inspector of levees and dredge ditches, St. Francis Drainage District; December, 1911, to April, 1912, Draftsman on drainage surveys; April, 1912, to June, 1913, Engineer and Assistant Superintendent for A. B. Wills and Sons, Drainage Contractors, on the inspection of drainage districts, making estimates, designing dredge boats, etc.; June to November, 1913, Field Engineer for James R. Rhyne, Assoc. M. Am. Soc. C. E., in charge of the preliminary survey for a 30 000-acre drainage district; November, 1913, to September, 1914, Resident Engineer for the Marion County Drainage District, supervising the construction of levees, open and title ditches, as well as the erection of a pumping station and the installation of two 30 000 gal. per min., electrically-driven centrifugal pumps; February to September, 1914, also Chief of a field party on a levee and ditch location survey for the South Quinch Drainage District; October, 1914, office work with the Morgan Engineering Company; November, 1914, to November, 1915, made a drainage survey and reported on 300-acre farm and drainage and irrigation investigations in the Southwest; from November, 1915, to the summer of 1918, when he entered the military training camp, Mr. Shipley was employed as Drainage Engineer, for the Texarkana Pipe Works, making drainage surveys, designs, estimates, and reports, closing contracts and superintending all the drainage construction work for the Company.

He entered the First Engineer Officers' Training School, at Camp A. A. Humphreys, Va., in June, 1918, where he died of influenza on October 11th, 1918.

He was married to Miss Helen Cook, of Texarkana, Ark., on June 27th, 1918, and is survived by his wife, and two brothers, Lieut.

* Memoir prepared by J. H. Brillhart, M. Am. Soc. C. E.

John I. Shipley, M. D., U. S. N., and Elvin M. Shipley, who was stationed at Great Lakes, Ill., in the Aviation Service of the United States Navy.

Of Mr. Shipley, L. L. Hidinger, M. Am. Soc. C. E., Vice-President of the Morgan Engineering Company, says, "He was connected with our firm only a short time, but we were impressed with his stability of character, steadiness, and reliability." Mr. G. R. Layne, of the Texarkana Pipe Works, writes of him as follows: "My acquaintance with Mr. Shipley dates from the time he entered my employ about four years ago. He was with us three years as Drainage Engineer, principally on agricultural work, and I esteemed him very highly, not only for the value of his services, but as a thorough gentleman."

Mr. Shipley was a man of high moral character, modest and unassuming. Owing to the fact that his father died when he was still young, he was thrown on his own resources, and the record that he made in his short life, shows that he was efficient and thorough, and that the Profession lost a valuable man by his death.

Mr. Shipley was elected an Associate Member of the American Society of Civil Engineers on October 9th, 1917.

EDGAR FIELD SMITH, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 17TH, 1918.

Edgar Field Smith, the son of the late Ransom Eugene and Anna Lydia (Grout) Smith, was born at Saxtons River, Vt., on December 25th, 1877. He received his early education in the public schools of his home town, and in Vermont Academy, and was graduated from Boston University in 1900 with the degree of A. B.

After his graduation Mr. Smith worked as a Draftsman for a few years with several firms about Boston, among which were the Century Machine Company, the John Stuart Company, of Wollaston, Mass., and Sheaf and Jaastad, Electrical Engineers, of Boston.

In September, 1902, he entered the Massachusetts Institute of Technology, and was graduated in June, 1904, with a degree of B. S. in Civil Engineering.

From June to October, 1903, he was an Assistant Engineer with H. S. Ferguson, M. Am. Soc. C. E., at Millinocket, Me., and from June to September, 1904, he served as a Transitman for the Boston Elevated Railway Company. From September, 1904, to June, 1905, Mr. Smith was employed by the United States Government at Parkersburg, W. Va., on the construction of Lock and Dam No. 18, Ohio River. In June, 1905, he became Designing Draftsman for the Bureau of Filtration, Pittsburgh, Pa.

Late in 1905 he left Pittsburgh to accept a position in New York City with the New York Central Railroad. Early in 1906 he was appointed Assistant Engineer in the Construction Department of the International Paper Company, of New York City.

In September, 1907, Mr. Smith became an Assistant Engineer and Designer with the New York Board of Water Supply, with which he remained up to the time of his death on October 17th, 1918, except for a period of ten months, from September, 1911, to July, 1912, when he was Assistant Professor of Civil Engineering at the Brooklyn Polytechnic Institute. With the Board of Water Supply he advanced to the position of Assistant Designing Engineer in charge of a squad, and had responsible charge of designing portions of the steel pipe siphons of the Catskill Aqueduct, of the connection chambers of the City Aqueduct Pressure Tunnel, of the City pipe lines, of the Narrows Crossing, and of the Eastview Filters.

In addition to his work with the Board of Water Supply, he was an Instructor in Surveying at the Brooklyn Polytechnic Institute for the extension classes from 1907 to 1918, and at the time of his death was about to take a similar position at Cooper Union.

* Memoir prepared by Arthur R. Holbrook, Assoc. M. Am. Soc. C. E.

Mr. Smith's work was marked by thoroughness, neatness, and system; he believed in doing well everything he undertook; he was very accurate, and possessed an excellent memory.

He was married on September 12th, 1907, to Miss Caroline Leicht, of Southold, Long Island. He is survived by his widow and one child, Edgar Field, Jr., two brothers, Walter Leroy Smith and R. Eugene Smith, of Malden, Mass., and one sister, Mrs. Della (Smith) Brett of Miami, Fla. He was very much devoted to his family.

Mr. Smith's death was very sudden; he fell a victim to the epidemic of influenza, followed by pneumonia, which swept this country in the latter part of 1918.

He was a member of the New England Water Works Association, and of the Beta Theta Pi Fraternity, which he joined while at Boston University.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on May 1st, 1907.

AUSTIN RUSSELL WILLARD SPERRY, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1918.

Austin Russell Willard Sperry, the only son of Austin B. and Katherine M. Sperry, of the family that organized the Sperry Flour Company, was born in Stockton, Cal., on December 26th, 1888.

He attended the Stockton schools, spent two years in the High School, and then went to Berkeley and studied at Boone's University School for two years. He was graduated in Civil Engineering from the University of California in 1910, having been a member of the G. E. K. High School Fraternity and the Alpha Delta Phi College Fraternity. He was a member of the 'Varsity crew and was also an accomplished 'cellist.

Immediately after his graduation, Mr. Sperry started work as an Inspector in the Maricopa oil fields, but soon moved to the Crocker-Huffman Irrigation Project, near Merced, and before the completion of this work he had risen to be Chief Engineer. He continued this specialty of irrigation engineering until his death. In 1912, he took hold of an irrigation unit for the Natomas Company of California and completed an extensive system. He then directed the operation and maintenance of the system, and also took charge of the works of the Folsom Water Company, looking after the water supply and retaining dams in connection with the fleet of gold dredges operated by this Company.

In March, 1917, he accepted the position of Supervising Engineer for the Anderson Cottonwood Irrigation District, in the Upper Sacramento Valley, and directed work involving an expenditure of $700 000. The many extensive and, in several cases, unique structures in this District, as well as the excellent character of the work, stand as monuments to Mr. Sperry's engineering ability and initiative.

In addition to his engineering activities, Mr. Sperry was associated in a financial way with the Rising River Ranch Company, which owns extensive stock ranges.

On September 25th, 1912, he was married to Gertrude Fancher, of Merced, Cal., who died on January 19th, 1917, leaving one daughter, Katherine. On July 12th, 1918, he was married to Marguerite Cron, of Berkeley, Cal., who, with his mother, survives him.

His integrity of character, lovable personality, and ability in handling men had endeared him to all with whom he came in contact, and the blow to his loved ones and friends was the more severe in that he

* Memoir prepared by R. G. Clifford, Assoc. M. Am. Soc. C. E.

was stricken suddenly, in the prime of life, with a brilliant future opening up in his chosen profession. He leaves a host of warm personal friends and business associates.

Mr. Sperry was elected a Junior of the American Society of Civil Engineers on December 6th, 1910, and an Associate Member on January 15th, 1917.

GEORGE HIPPESLEY STANLEY STEPHENS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1918.

George Hippesley Stanley Stephens, the eldest son of Frederick Orlando Stephens, now of The Homestead, Orangezicht, Cape Town, South Africa, was born in Norwich, England, on November 20th, 1882. He was educated at Tunbridge Wells, England, and at the Diocesan College, South Africa. He was articled to his father, a Civil Engineer, from 1900 to 1905.

From 1901 to 1908, Mr. Stephens was engaged as Field Assistant and Assistant Engineer by Messrs. G. Pauling and Company, Limited, of London and Cape Town, on the surveys and construction work in connection with the extension of the New Cape Central Railway to Mossel Bay, in Cape Colony, South Africa, and the Stuartstown Railway in Natal. In 1908 and 1909, he was employed by the same firm as Assistant Engineer on the survey of the Bwana Mkubwa ("Star of Congo") Railway, from the Bwana Mkubwa copper mine, in Northwestern Rhodesia, to the "Star of the Congo" Mine, in the Congo State.

In 1909 and 1910, Mr. Stephens served as Assistant Engineer on the construction of the Bwana Mkubwa and Broken Hill Railway, in Northwestern Rhodesia, and from that time until the early part of 1911, he was engaged as First Assistant Engineer on the survey of the Mazoe Railway, also in Rhodesia.

From 1911 to 1917, he was employed by the South African Railway Administration on the construction of the railway extension from Caledon, Upington, and Carnarvon, and, from that time until his death which occurred at Paarl, South Africa, on October 13th, 1918, Mr. Stephens was engaged on various surveys and investigations for irrigation projects for the Department of Irrigation in the Union of South Africa.

Mr. Stephens' untimely death, from pneumonia supervening influenza, at the age of 36, came as a painful shock to his professional associates and to his wide circle of friends in many parts of Southern Africa. To all of them he was the soul of friendship, and there must still be many who, in the course of the lonely journey by which entry to the Congo from the south was made before the construction of the railway was completed, can recollect his warm welcome and generous hospitality on their way through the illimitable veldt. His largeness of heart indeed was his chief characteristic, and, with it, there went a cheerfulness and constancy that braved every trial, shining even more brightly when the final command came for him, too, to "pass over".

* Memoir prepared from information on file at the Headquarters of the Society.

The thorough training which Mr. Stephens had enjoyed under all classes of South African conditions, had eminently fitted him for the highest positions in the service to which he was finally attached, and his early death will prove a loss to the Union in which he had made his home.

Mr. Stephens was married in 1916 to Grace Winifred Taylor, the only daughter of Charles Taylor, of Paarl, who survives him after a brave struggle with the epidemic which, in October, 1918, swept through South Africa.

Mr. Stephens was elected an Associate Member of the American Society of Civil Engineers on December 5th, 1911. He was also an Associate Member of the South African Society of Civil Engineers.

EDWARD PORTER ALEXANDER, Jun. Am. Soc. C. E.*

DIED SEPTEMBER 5TH, 1918.

Edward Porter Alexander was born at Duluth, Minn., on November 4th, 1891. He was a lineal descendant of Gen. E. P. Alexander, of Georgia, a graduate of West Point and an officer in the Civil War.

Edward Porter Alexander attended school in Duluth and then entered the University of Minnesota, after which he received his engineering education at the Massachusetts Institute of Technology, from which he was graduated in 1914 with the degree of B. S. in Civil Engineering. He returned to Duluth where he formed a partnership with Mr. James A. Farrell, covering the field of general surveying, municipal, railroad, and mining engineering.

At the beginning of the trouble with Mexico, Mr. Alexander, while still at the Institute of Technology, made application for a commission in the army; and, when the United States entered the World War, he received his commission as First Lieutenant in the Engineers Reserve Corps. He entered the training camp at Fort Snelling, Minn., was afterward transferred to the Engineers Training Camp at Leavenworth, Kans., and from there was sent to Fort Travis, Tex., for overseas training. He was with the 509th Engineers at St. Nazaire, France, as Adjutant, up to the time of his death.

Mr. Alexander was a very capable and energetic man, of high character and pleasing personality. He was married on August 20th, 1917, to Miss Myra Salyards, of Duluth, who survives him.

Mr. Alexander was elected a Junior of the American Society of Civil Engineers on November 4th, 1914.

* Memoir prepared by Lyonel Ayres, M. Am. Soc. C. E.

VICTOR HUGO BELL, Jun. Am. Soc. C. E.*

DIED JANUARY 6TH, 1919.

Victor Hugo Bell was born at Hutchinson, Kans., on July 6th, 1891. When he was only a few years old, his mother moved to San Diego, Cal., where the boy received his early schooling. He began his engineering studies at the University of Washington, and, in 1912, entered Valparaiso University at Valparaiso, Ind., from which he was graduated with the degree of B. C. E. in June, 1914, receiving his C. E. degree in August of the same year. At college, Mr. Bell had the reputation of being an honest, energetic, and apt student.

Prior to entering college, he was employed from March to November, 1910, as Stakeman, Chainman, and Leveler with the Chicago, Milwaukee and Puget Sound Railway Company; and from December, 1911 to March, 1912, and from March to June, 1912, as Transitman with L. B. Harris, Consulting Engineer, of San Diego, Cal., and the Western Salt Company, of Otay, Cal., respectively.

Immediately after his graduation in 1914, Mr. Bell returned to California and opened an office in Calexico for the private practice of engineering. His work there was chiefly in connection with the irrigation of the Imperial Valley, such as erecting timber headgates and bulkheads for the South Alamo Water Company, irrigation, land grading, and the subdivision of 169 acres into city lots, for the Royal Lands Company, as well as assisting the City Engineer of Calexico and working on numerous other engineering and surveying jobs.

When the United States entered the World War, Mr. Bell closed his office in Calexico and enlisted in the Army. He entered the Science and Research Division and was graduated as a Second Lieutenant from the United States Army School in Texas. He was then assigned to the Science and Research Division of the Army Balloon School at Arcadia, Cal., where he was stationed at the time of his death. He was killed on January 6th, 1919, in an automobile accident on the highway between Pasadena and Los Angeles, and was buried with full military honors at Fort Rosecrans, Cal. He is survived by his mother, Mrs. Mary F. Bell.

Lieut. Bell was a Member of the Western Society of Engineers and was prominent as a Knight of Columbus. He was elected a Junior of the American Society of Civil Engineers on March 14th, 1916.

* Memoir prepared from information on file at the Headquarters of the Society.

DEAN CHASE, Jun. Am. Soc. C. E.*

DIED MAY 26TH, 1919.

Dean Chase, the son of Clinton Alden Chase and Sallie Harmon Chase, was born at St. Louis, Mo., on May 5th, 1890. After having been graduated from the McKinley High School, at St. Louis, in 1909, he spent one year in the Engineering Department of the Missouri Pacific Railway. In 1910, he entered the University of Illinois, from which he was graduated in 1914, as a Municipal and Sanitary Engineer, with the degree of B. S.

After his graduation, Mr. Chase entered the employ of the Water Department of the City of St. Louis, and during parts of 1914 and 1915, was engaged at the Chain of Rocks on the construction of the new filtration plant.

On the completion of this work in the spring of 1915, he concluded to enter the commercial field, and secured a position with the Certainteed Products Corporation as Traveling Salesman, with headquarters at New Orleans, La. Mr. Chase was signally successful in his new endeavor and continued at this work until shortly after the declaration of war with Germany, when he entered the First Officers' Training Camp, at Fort Logan H. Roots, near Little Rock, Ark. After he had been there several weeks, he was rejected for trivial physical defects. Although greatly disappointed, he was undaunted in his zeal to go to his country's aid, and in August, 1917, he enlisted again at New Orleans, and soon thereafter entered the Second Officers' Training Camp, at Fort Winfield Scott, Cal.

On November 27th, 1917, Mr. Chase was commissioned a First Lieutenant in the Coast Artillery Corps, and was assigned to the 62d Regiment, with which he was identified until he was honorably discharged from the service at Camp Custer, Mich., on March 8th, 1919. He saw service with his regiment in France from July 31st, 1918, to February 6th, 1919, but did not get to the front, although the regiment was about to be ordered there when the Armistice was signed. On account of his appointment as Regimental Orienteur, Lieut. Chase became a Staff Officer and was twice recommended for promotion, but, here, again, the signing of the Armistice prevented fulfillment. On May 26th, 1919, the evening of which he met his death, he was commissioned First Lieutenant, Coast Artillery Section, Officers' Reserve Corps of the U. S. Army.

From about March 20th, 1919, to the time of his death on May 26th, 1919, Lieut. Chase was in the employ of the Certainteed Products Corporation as traveling representative, with headquarters at Syracuse, N. Y. On the evening of the fateful day, while motoring

* Memoir prepared by C. W. Roehrig, Esq., St. Louis, Mo.

near Utica, N. Y., with two prospective customers, they met with an accident, the machine being overturned, and Lieut. Chase was killed instantly. He was buried from the family residence in St. Louis, Mo., on May 30th, 1919, with military honors.

Lieut. Chase possessed an analytical mind and the power of concentration to a high degree, and this was reflected in his student and professional life. He had a most promising career before him, and his untimely taking away will be mourned by a large circle of friends. He had led an honest, upright, and morally clean life, and was loved and respected by all who knew him.

Lieut. Chase was a Member of the Engineers' Club of St. Louis, and was elected a Junior of the American Society of Civil Engineers, on June 3d, 1915.

ERNEST CHARLES DEDICKE, Jun. Am. Soc. C. E.*

DIED JULY 15TH, 1918.

Ernest Charles Dedicke was born on February 19th, 1889, in the village of Dolgeville, Herkimer County, N. Y. His early childhood days were spent in the place of his birth, and it was here that he received most of his grammar school education. At the age of twelve, he moved with his family to Hamburg, Germany, where he remained until 1905, and where he completed his high school education.

On his return to the United States, the young lad, led by the same adventurous spirit that brought him back to his native shores, and prompted by a rare love of country which had become the more pronounced during his stay on foreign soil, joined the United States Army. He was assigned to the 11th U. S. Cavalry, and, with that regiment, took part in the campaign of Cuban Pacification, in 1906-08. He was honorably discharged in 1908, having reached the grade of Sergeant, which, in the old army, was a rare achievement for one of his years.

Mr. Dedicke returned to the States and took up the study of expert accountancy at Valparaiso University. While employed as Assistant at the Remington Works, at Ilion, N. Y., his health failed, and he was compelled to turn to the life of the great outdoors. His choice fell on the Engineering Profession. While gathering practical knowledge, he acted as Foreman on the construction of a large dam at Dallas, Tex., also on another at Hinkley, N. Y., and was connected with several highway improvement contracts in various parts of New York State. Besides thus gaining practical experience, he was diligently studying with the American School of Correspondence. In 1913, Mr. Dedicke was appointed Junior Engineer under the Civil Service System of the City of New York, and was employed on the construction of the New York City Subways. In 1914, he was transferred to the Utica Office of the New York State Highway Department, where he remained until he once more joined the colors. This was at the time of the Mexican trouble.

He enlisted in Troop G, of the 1st New York Cavalry, at Utica, N. Y., with which he saw service on the Mexican border until the early spring of 1917. He then received a commission as Second Lieutenant of Engineers, and was ordered to Madison Barracks, N. Y., for training. Three months later, he was assigned to the 6th U. S. Engineers, at Washington Barracks, D. C., and with that regiment left for France in November, 1917. This outfit did splendid work as an independent unit with the British Army. Company D, of which Lieut. Dedicke was First Lieutenant, was decorated for bravery by the British Government, for heroic work with the now famous emergency army of

* Memoir prepared by O. J. Dempster, M. Am. Soc. C. E.

Gen. Carey, which army held back the German advance in March, 1918, until reinforcements arrived. In April, the 6th Engineers were assigned to the Third Division, and with the Second Division won immortal fame at Chateau Thierry, in June and July, 1918. In July, Lieut. Dedicke was assigned as Engineer Officer with the 4th U. S. Infantry, and as such had charge of all trench work and barbed wire entanglements.

It was while directing the construction of these works, 5 km. east of the now famous French town of Chateau Thierry, and during the early morning German barrage of July 15th, 1918, that Lieut. Dedicke was mortally wounded by shrapnel, dying at 5 o'clock that afternoon, in Field Hospital No. 27.

He was beloved by his men and respected by his superiors, and was a finely trained engineering officer. He is sleeping the eternal sleep among his fellow heroes in Cemetery No. 284, near the spot where the ambitions of the Hun were stayed.

Lieut. Dedicke was elected a Junior of the American Society of Civil Engineers on November 28th, 1916.

DONALD POWER DENHAM, Jun. Am. Soc. C. E.*

DIED OCTOBER 23d, 1918.

Donald Power Denham, the son of Mr. and Mrs. W. H. Denham, was born at Akron, Ohio, on June 14th, 1890. In 1904, the family moved to Peterborough, Ont., Canada, and the boy prepared for college at the Peterborough Collegiate Institute. In 1910, he entered Cornell University, from which he was graduated in 1914 with the degree of Civil Engineer.

In April, 1915, Mr. Denham entered the employ of the Leonard Construction Company, and was engaged on work for that Company, as follows: Until April, 1916, as Field Engineer on the construction of a reinforced concrete ore bin and a reinforced concrete steel mill building for the New Jersey Zinc Company, at Franklin, N. J.; from April to October, 1916, as Resident Engineer on the construction of a garbage plant for the Metropolitan By-Products Company, on Staten Island, New York; and from October to December, 1916, on the construction of a reinforced concrete building for the Douglas Starch Company, at Cedar Rapids, Iowa.

In December, 1916, Mr. Denham's services were loaned by the Leonard Construction Company to the Quaker Oats Company for special drafting and designing in connection with the construction of the latter's plant at Cedar Rapids, Iowa, and from February to May, 1917, he was engaged as Field Engineer on the construction of reinforced concrete buildings for the Sinclair Oil Company, at Kansas City, Mo. Later, he went to Peterborough, Ont., Canada, as Engineer in Charge for the Quaker Oats Company on the construction of its immense plant at that place. He was also engaged at Saskatoon, Sask., Canada, on mill construction for the same Company.

When war was declared, Mr. Denham made repeated efforts to get into military service, but owing to a severe illness a short time before, he was unable to pass the physical examination. While working at Saskatoon, he received a Government appointment with the United States Bureau of Yards and Docks, at Charleston, S. C., and was on his way to that place when he was taken ill with pneumonia from which he died on October 23d, 1918.

While at Cornell Mr. Denham was a member of the Sigma Nu, Semaphore, and Glee Clubs, and he was also a member of the *Civil Engineer* Board.

Mr. Denham was elected a Junior of the American Society of Civil Engineers on April 17th, 1917.

* Memoir prepared from information on file at the Headquarters of the Society.

RALPH RICHARDSON MARRIAN, Jun. Am. Soc. C. E.*

DIED OCTOBER 17TH, 1918.

Ralph Richardson Marrian was born at Watertown, N. Y., on March 24th, 1890. He received his early education in the Watertown schools, and then attended Cornell University, from which he was graduated with the degree of Civil Engineer in 1915.

During the summer of 1914 Mr. Marrian was a Transitman in the Engineering Department of the New York Central Railroad, and, in June, 1915, was employed by Messrs. Hazen, Whipple and Fuller, in New York City. While so engaged, he enlisted in the Seventh New York Infantry, and, in the summer and fall of 1916, served with that regiment on the Mexican border. On the return of the regiment, he was discharged.

When the United States entered the World War, Mr. Marrian re-enlisted and received a commission as Second Lieutenant. After two months of training at Camp American University, in Washington, D. C., he was assigned to the 105th U. S. Engineers, then stationed at Camp Sevier, Greenville, S. C. In May, 1918, he went overseas with that regiment, and soon afterward was promoted to the rank of First Lieutenant.

Lieut. Marrian was wounded on October 17th, 1918, by a bursting shell while repairing roads under shell fire near Molain, northeast of Bohain, in France, in order that artillery might follow up an infantry drive. A shell burst near him and his men, killing two outright and wounding four besides himself. He died soon after he was carried to the main dressing station, and was buried in the Military Cemetery at Montbrehain, where his company built a fence around the grave and placed a cross at its head.

Col. Joseph Hyde Pratt, commanding the 105th Engineers, in reporting his death to his family, wrote that Lieut. Marrian had been doing splendid service, and added: "He was brave, conscientious, and a good leader. He had the confidence of his men, and he and they gave the best they had for their country."

Lieut. Marrian was elected a Junior of the American Society of Civil Engineers on April 18th, 1916.

* Memoir prepared by Woodford Patterson, Esq.

RICHARD JETER PALMER, JR., Jun. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1918.

Richard Jeter Palmer, Jr., was born at Harmony Village, Va., on August 10th, 1887. He finished his course at West Point Academy in the spring of 1903, and, in the fall of that year, entered Washington and Lee University, where he remained until 1905. Then he entered the Virginia Polytechnic Institute, and was graduated in 1907 with the degree of Bachelor of Science.

From August, 1907, until June, 1908, was employed as Draftsman by the American Bridge Company, at Trenton, N. J., and then opened an office at West Point, Va., doing private engineering work and land surveys, sewer and highway construction being a specialty. In November, 1909, he entered the employ of the Gulf, Colorado and Santa Fé Railway. During the 19 months he was in the service of this road he had several promotions. He then accepted a position with the Richmond Cedar Works, as Instrumentman, making land, timber, and drainage surveys.

In April, 1912, Mr. Palmer entered the service of the Norfolk and Western Railroad, as Rodman, and, after 3 months, he was made Inspector on the construction of a twenty-one stall round-house, turntable, machine shop, blacksmith shop, oil house, storehouse, coaling station, 100 000-gal. reinforced concrete tank, ash hoist, and minor auxiliaries. After completing this work he became Instrumentman on maintenance of way, making yard layouts and estimates.

Mr. Palmer was appointed Resident Engineer on the construction of two steamer warehouses, 800 and 1 200 ft. long, respectively, and each 222 ft. wide, the development cost being $1 800 000. During the last 18 months he was on railroad construction at Camp Lee, Va., and the rearrangement of Coal Pier No. 3, at Norfolk, Va.

In his professional career, Mr. Palmer developed those qualities which insure success. Owing to his fine qualities for thoroughness and as a man he won the esteem of all with whom he came in contact.

He was married to Miss Edyth Atwood, of Appomattox, Va., on October 15th, 1912.

On October 2d, 1918, he contracted influenza which developed into pneumonia and caused his death on October 13th. He is survived by his wife, mother, brother, and two sisters.

Mr. Palmer was elected a Junior of the American Society of Civil Engineers on October 10th, 1916.

* Memoir prepared by Loomis L. Kelly, Esq.

CHARLES GILBERT REILLY, Jun. Am. Soc. C. E.*

DIED SEPTEMBER 29TH, 1918.

Charles Gilbert Reilly, the son of the late John C. Reilly and Ursula O'Connor Reilly, was born at Pittsburgh, Pa., on December 12th, 1889. After his preliminary schooling, Mr. Reilly entered Princeton University, from which he was graduated in June, 1912, with the degree of Civil Engineer.

After his graduation, Mr. Reilly traveled in Europe for several months, and, on his return to Pittsburgh, entered the employ of the Pittsburgh Railways Company, on double-track work from Pittsburgh to Castle Shannon, on the Castle Shannon Division. He was also engaged, for a short time, as Engineer with the Philadelphia Company, of Pittsburgh.

In 1915, he was appointed Superintendent of Construction on the plant of the Pittsburgh Model Engine Company, of which company he was also a Director and Assistant to the President.

In May, 1917, one month after war with Germany was declared, Mr. Reilly entered the First Officers' Training Camp at Fort Niagara, N. Y., and was commissioned a First Lieutenant. In August, 1917, he was assigned to Company D of the 313th Infantry, then stationed at Camp Meade, Md., where he remained until he sailed for France with the Seventy-ninth Division in July, 1918. Lieut. Reilly had been in France only one month when he was promoted to be Captain of his company.

With the Seventy-ninth Division, his company saw hard service on the fighting line in France, and Capt. Reilly was killed in action on September 29th, 1918, during the drive in the Argonne Forest.

Capt. Reilly was of an amiable and happy disposition, with a smile for every one, and had made for himself many friends. During his college days, he was Assistant Manager and Manager of the baseball team at Princeton and a member of the Tiger Inn Club. He was also a member of the Pittsburgh Athletic Association and the Oakmont Country Club.

Capt. Reilly was elected a Junior of the American Society of Civil Engineers on May 7th, 1913.

* Memoir prepared from information on file at the Headquarters of the Society.

HUGO S. SPEIDEL, Jun. Am. Soc. C. E.*

DIED JUNE 17TH, 1919.

Hugo S. Speidel was born at Rock Island, Ill., on September 3d, 1866. He obtained his early education in the public schools of that city, and, in 1888, received the degree of Civil Engineer from the University of Illinois.

Mr. Speidel began his engineering career, in July, 1885, as Chainman and Rodman on the construction of the St. Joseph and Iowa Railroad (Rock Island Lines), near St. Joseph, Mo. In June, 1886, he was transferred to Kansas, and was employed on the construction of the Chicago, Kansas, and Nebraska Railroad (also Rock Island Lines), serving first as Draftsman in the general office and, later, as Assistant in charge of a 10-mile division at Sabetha, Kans. After the completion of this division in January, 1887, Mr. Speidel was appointed Transitman on the location of the line through Kiowa County, Kansas, and when the location work was finished, he was given charge of the construction, first of a 10-mile division, and, later, of a 20-mile division, of the line. About July, 1887, he was placed in charge of the construction of all bridges in Kiowa County.

After the completion of this work in September, 1887, Mr. Speidel left the railroad service to enter the University of Illinois, in order to complete the senior year of his college course and obtain his degree.

Immediately after his graduation, he moved to Paterson, N. J., and in July, 1888, entered the employ of the Passaic Bridge Company as Civil Engineer.

In 1898, Mr. Speidel purchased an interest from Mr. David Henry, of Paterson, N. J., in a general building business, and in June, 1900, the David Henry Building Company was incorporated with Mr. Speidel as President, which position he held at the time of his death on June 17th, 1919.

About 1904, Mr. Speidel removed from Paterson and established his residence in Saddle River Township, Bergen County, N. J., and, in 1916, when the Borough of East Paterson was incorporated, he was appointed its first Mayor, which office he held until his death.

He was married on December 20th, 1893, to Adaline S. Speidel, of Urbana, Ill., and is survived by a daughter, Marion W. Speidel.

Mr. Speidel was elected a Junior of the American Society of Civil Engineers, on February 5th, 1890.

* Memoir prepared from information on file at the Headquarters of the Society.

JAY WILLIAM TOMS, Jun. Am. Soc. C. E.*

DIED DECEMBER 20TH, 1918.

Jay William Toms was born at Frederick, Md., on September 16th, 1888. He prepared for college at the Frederick High School, entering Cornell University in 1905 and receiving the degree of C. E. in 1909. In college he w a member of Phi Kappa Sigma Fraternity, Semaphore, an honorary C l Engineering Society, and the Maryland Club. He was President f the Cornell Association of Civil Engineers in his Senior year.

Following his graduation, Mr. Toms was, until 1916, a designer of reinforced concrete work for the Corrugated Bar Company, first in St. Louis, Mo., then in turn in its Buffalo, New York City, New Orleans, and Chicago offices, except for about a year, when he was employed as Estimator in the Atlanta Office of the Industrial Engineering Company.

In 1916, he formed a partnership with John Benedict, of Davenport, Iowa, for the sale of building materials, under the firm name of The John Benedict Company. In the fall of 1917, Mr. Toms' health failed, due to overwork, resulting in a severe case of pneumonia from which he never entirely recovered. In the spring of 1918 he went to Denver, hoping to improve his condition, but was recalled in October by the death of his brother, Keefer, from influenza. Having been appointed Executor of his brother's estate, he postponed his return to Denver, against the advice of his physician. The eastern climate was too severe for him, and, after contracting influenza, he lived only five days.

Mr. Toms' lovable disposition had endeared him to a host of friends, who were shocked to learn of his untimely death. He is survived by one brother, Captain Raymond E. Toms, of the Field Artillery, U. S. A.

Mr. Toms was elected a Junior of the American Society of Civil Engineers on November 30th, 1909.

* Memoir prepared by A. P. Haney, Assoc. M. Am. Soc. C. E.

TRANSACTIONS

OF THE

American Society of Civil Engineers

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